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CENTRE DE RECHERCHES MINERALES

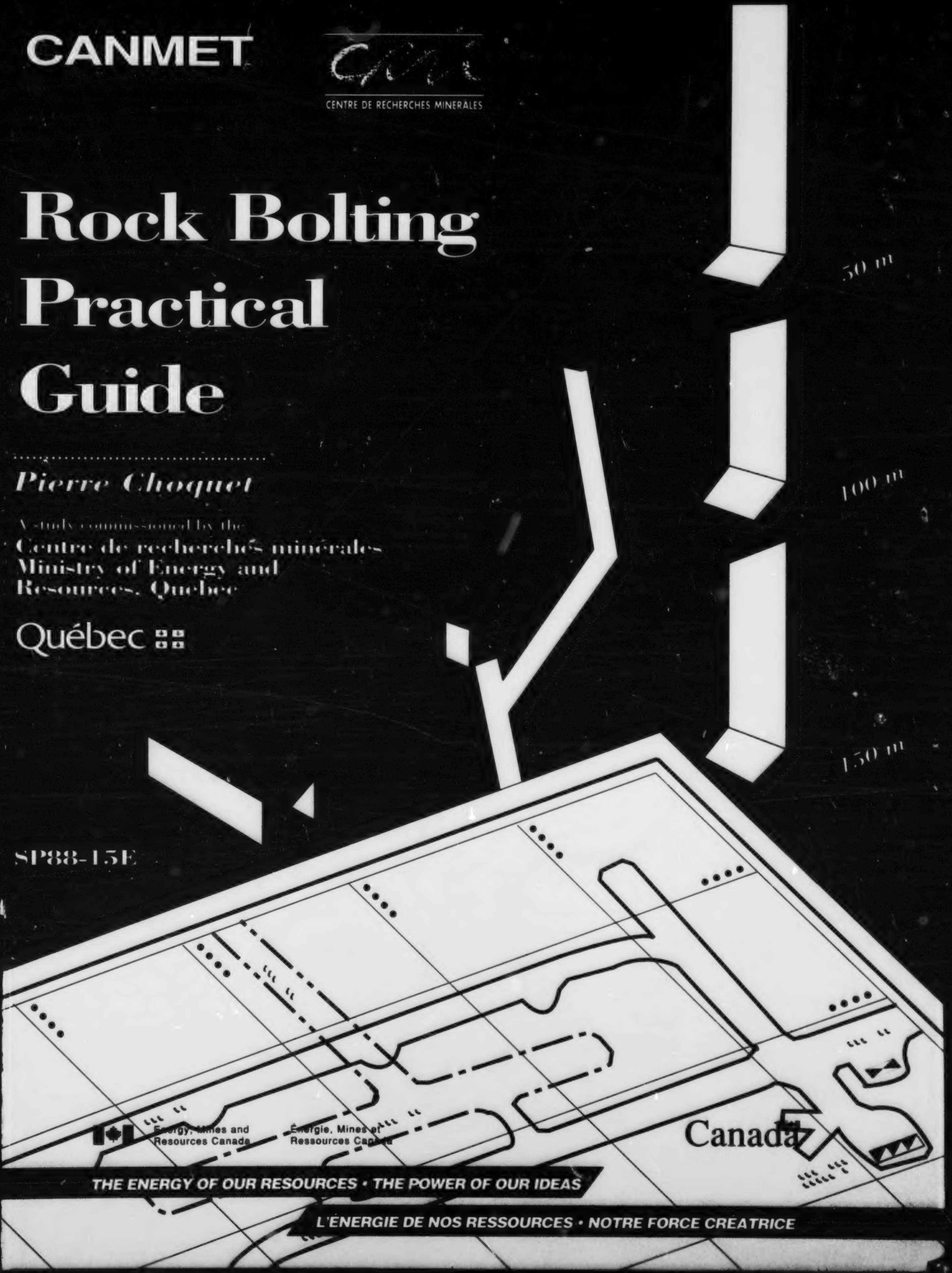
Rock Bolting Practical Guide

Pierre Choquet

A study commissioned by the
Centre de recherches minérales
Ministry of Energy and
Resources, Quebec

Québec

SP88-15E



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Available in Canada through

Associated Bookstores
and other booksellers

or by mail from

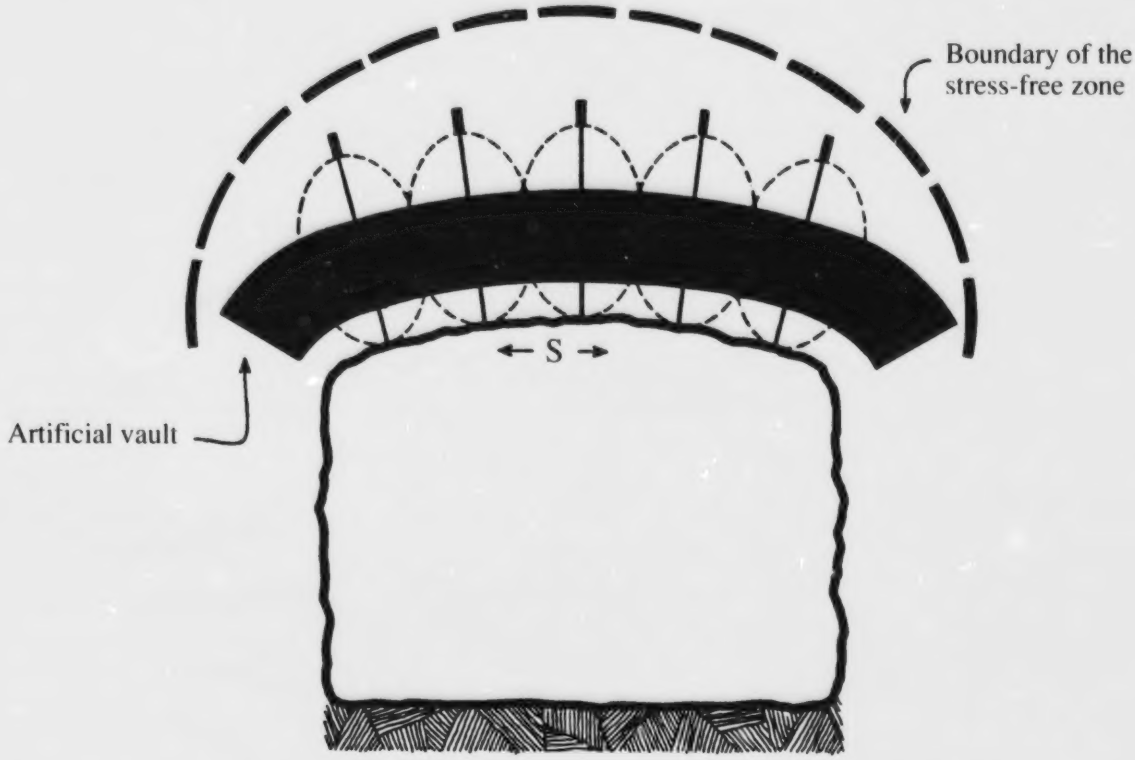
Canada Communication Group — Publishing
Ottawa, Canada K1A 0S9

Catalogue No. M38-15/88-15E
ISBN 0-660-13994-4

91-05150/

Scientific authority: Jean-Marc Robert, Centre de recherches minérales

English translation: CANMET


Boundary of the stress-free zone
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Artificial vault

Rock Bolting Practical Guide

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G1K 7P4

210 pp

MICROMEDIA

FOREWORD

In January 1987, the Centre de recherches minérales (CRM) of the Government of Quebec published Dr. Pierre Choquet's handbook *Guide d'utilisation du boulonnage* (*Rock Bolting Practical Guide*). The handbook was the result of a three-year study sponsored by the CRM and carried out by the CRM and Université Laval. The Ground Control Committee of the Association minière du Québec inc. acted as an advisory committee to the project.

The handbook, originally published in French, has been very well received by the mining industry in Quebec. There has been a strong demand for an English edition, and the assistance of the Canada Centre for Mineral and Energy Technology (CANMET) in sponsoring a translation was requested.

We have been very pleased to do so, and, because the document addresses one of the topics in a new series of publications on underground metal mining, we have published it as a part of that series.

John E. Udd

John E. Udd
Director
Mining Research Laboratories
CANMET

AVANT-PROPOS

En janvier 1987, le Centre de recherches minérales (CRM) du gouvernement du Québec a publié le *Guide d'utilisation du boulonnage*, préparé par M. Pierre Choquet. Ce guide est le fruit de trois ans d'études parrainées par le Centre de recherches minérales et menées par le CRM et l'Université Laval. Le Comité du contrôle de terrain de l'Association minière du Québec inc. revêtait le rôle de conseiller du projet.

Le guide original, publié en français, a été très bien accueilli par l'industrie minière du Québec. En raison de la demande importante d'une version anglaise du guide, la participation du Centre canadien de la technologie des minéraux et de l'énergie (CANMET) a été sollicitée pour la traduction du document.

Nous avons été très heureux d'accepter cette invitation, puisque le guide traite d'un des sujets d'une nouvelle série de publications sur l'exploitation souterraine de mines de métaux. Le présent guide est donc publié dans le cadre de cette série.

John E. Udd

John E. Udd
Directeur
Laboratoires de recherche minière
CANMET

PREAMBLE

The Centre de recherches minérales is pleased to offer you the *Rock Bolting Practical Guide*, prepared for operators of underground mines in the hard- and fissured-rock formations of eastern Canada. This document is the result of three years of studies and research carried out in Quebec mines by the Centre de recherches minérales and Université Laval.

The purpose of this document is to help mining operators improve the quality of the support in their underground excavations by making the most of rock bolting and its elements. Many different activities were necessary to carry the development of this guide to completion: a preliminary investigation of rock bolting guidelines; a survey and measuring campaign on the use of rock bolting in Quebec mines; meetings and discussions with the Ground Control Committee of the Association minière du Québec inc.; a visit to manufacturers of rock bolting equipment; and foreign missions.

This document forms part of the list of guides that the Centre de recherches minérales is in the process of producing and distributing for the benefit of mining operators. We believe that they will find this guide useful because it summarizes the most recent knowledge and applications of rock bolting and may possibly help reduce the frequency of rock falls and accidents caused by them.

Marc Denis Everell*
Director General, Centre de
recherches minérales

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* Marc Denis Everell is now Assistant Deputy Minister of the Mineral and Energy Technology Sector of Energy, Mines and Resources Canada.

PRÉAMBULE

Le Centre de recherches minérales est heureux de vous présenter ce *Guide d'utilisation du boulonnage* destiné aux exploitants souterrains de roches dures et fissurées qu'on retrouve dans l'Est du Canada. Ce document est le fruit de trois ans de travaux et de recherches menés dans les mines du Québec par le Centre de recherches minérales et l'Université Laval.

Le but de ce document est d'aider l'exploitant minier à améliorer la qualité du soutènement de ses excavations souterraines par une meilleure utilisation du boulonnage et de ses composantes. Un éventail d'activités fut nécessaire pour mener à bien la réalisation de ce guide : une enquête préalable sur les règles d'utilisation du boulonnage; une campagne de relevés et de mesures sur le boulonnage dans les mines du Québec; des rencontres et des discussions avec le Comité du contrôle de terrain de l'Association minière du Québec inc.; une visite des fabricants de matériel de boulonnage; des missions à l'étranger.

Ce document s'inscrit dans la liste des guides que le Centre de recherches minérales est à produire et à diffuser pour le bénéfice des exploitants miniers. Nous croyons que ceux-ci trouveront intérêt à utiliser ce guide qui présente l'état des connaissances et des pratiques les plus récentes sur le boulonnage et qui permettra, nous l'espérons, de réduire la fréquence des chutes de blocs et des accidents qui en résultent.

Marc Denis Everell*
Directeur général, Centre de
recherches minérales

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* Marc Denis Everell est présentement sous-ministre adjoint du Secteur de la technologie des minéraux et de l'énergie d'Énergie, Mines et Ressources Canada.

AUTHOR'S FOREWORD

The *Rock Bolting Practical Guide* is the culmination of a series of investigations into ground reinforcement undertaken between 1983 and 1986.

This project was initiated by the Centre de recherches minérales (CRM), which identified the need for this work and subsequently helped in the definition of the general guidelines of the project. In collaboration with the CRM, an invitation was extended to the Quebec Mining Association (QMA) to have its newly formed Ground Control Committee act as an advisor to the project.

Judging by the warm welcome that I have received at all the mines visited during the initial phase of the project, and the subsequent assistance given to Mr. François Charette (at the time a post graduate student in the Department of Mines and Metallurgy) during the field work he conducted in 1985, the preparation of the guide generated a significant amount of interest in the Quebec mining community.

At the present time, there is a renewed interest in the further integration of rock mechanics principles into mining operations. The *Rock Bolting Practical Guide* is intended to be a "first-line" tool in that it introduces rock mechanics methodology into the solution of basic mining problems, such as the support of stopes and galleries. In reality, the guide concentrates more on the methods and practices of ground control rather than fundamental rock mechanics principles. The emphasis is on meeting the needs of the ground control practitioner. Consequently, the need for a mining specialist dedicated to ground control, as advocated by the QMA, is evident. This will allow for an interaction between miners, mine supervisors and the ground control engineer whereby each party will contribute to the flow of information and the ground engineer will educate the work force on the significance of the different ground reinforcement measures. It is felt that an educated work force has a lot to contribute to the long-term ground control program of a mining operation.

The impact of an increased awareness of ground control techniques is multi-faceted. First, it will contribute to a decrease in the number and gravity of accidents resulting from rock falls. Moreover, it is hoped that it will have an effect on the cost of rock bolting, primarily by ensuring that each bolt installed performs the task that it was designed to do, and that it is the optimum reinforcement tool for the encountered ground conditions. At the same time, by ensuring that a proper installation has been done, it is possible to reduce the costs of rebolting gallery sections where the initial support was inappropriate or has proved inadequate.

A great number of people, too many to list individually, have helped make this guide a reality. The list includes mine operators, research scientists and equipment manufacturers. Their contribution and assistance is gratefully acknowledged. The field portion of this work was undertaken with the assistance and cooperation of the following mines: Bousquet, Camflo, Corbet, Dest-or, Gaspé, Kiena, Lac Shortt, Montauban, Niobec, Opemiska. Special acknowledgements are also directed to Mr. François Charette for his outstanding field work, Mr. Ivan E. Hugo, formerly with Eimco Secoma, for his assistance in writing the chapter on mechanized rockbolting, and Mr. Jean-Marc Robert, division director at CRM, and Mr. Louis Bienvenu, who is responsible for ground control activities at CRM, for their advice and support throughout this project.

In closing, I would like to express the hope that the English version of the guide will generate the same level of discussion and deliberation as the original French version did. The English version differs in a small way from the French version in that it incorporates recent technological developments that have occurred. This was done to ensure that the guide is up to date. More specifically, the last part of Chapter 4, on the methods for selecting the length and spacing of rock bolts, was rewritten and an additional chapter on mechanized rock bolting was introduced. The scope and spirit of the guide was left intact.

P Choquet

Pierre Choquet

AVANT–PROPOS DE L'AUTEUR

Le *Guide d'utilisation du boulonnage* est le résultat d'une série de recherches sur la consolidation des terrains menées de 1983 à 1986.

Ce projet a été lancé par le Centre de recherches minérales (CRM), qui a reconnu le besoin d'entreprendre ce genre de travail et qui, par la suite, a aidé à en définir les grandes lignes. C'est également avec la collaboration du CRM qu'a été invité le Comité du contrôle de terrain de l'Association minière du Québec (AMQ) à agir en qualité de conseiller du projet.

La préparation du guide a suscité un vif intérêt dans la communauté minière du Québec, si j'en juge par l'accueil chaleureux qui m'a été réservé à toutes les mines que j'ai visitées au cours de la phase de démarrage du projet, et comme en témoigne l'aide accordée à François Charette (alors étudiant de troisième cycle au Département de mines et métallurgie) au cours des travaux qu'il a menés sur le terrain en 1985.

Cet intérêt s'explique probablement aussi par le renouveau que connaît actuellement l'intégration plus poussée des principes de la mécanique des roches aux opérations minières. Le *Guide d'utilisation du boulonnage* se veut un outil de premier rang puisqu'il recourt à la méthodologie de la mécanique des roches pour résoudre le problème minier de base qu'est le soutènement des chantiers et des galeries. En réalité, il traite plutôt des méthodes et des pratiques de contrôle des terrains plutôt que des principes fondamentaux de la mécanique des roches, car il cherche surtout à répondre aux besoins du praticien du contrôle des terrains. En conséquence, le contrôle des terrrains devrait être placé sous l'autorité d'un spécialiste, ce que recommande d'ailleurs l'AMQ. Il pourrait ainsi y avoir des discussions et des échanges d'informations entre les mineurs, les surveillants de mine et l'ingénieur responsable du contrôle des terrains. La tâche de ce dernier serait de renseigner les employés sur l'importance des diverses mesures visant à consolider le terrain et, forts de ces connaissances, ces employés pourraient à long terme apporter une contribution efficace au programme de contrôle des terrains dans une exploitation minière.

Une meilleure compréhension des techniques de contrôle des terrains aura des incidences sur de nombreux plans. Premièrement, elle contribuera à une réduction du nombre et de la gravité des accidents attribuables aux chutes de terrain. En outre, elle influera vraisemblablement sur le coût du boulonnage, principalement en donnant l'assurance que chaque boulon posé est utile et qu'il représente l'outil de consolidation le mieux approprié aux conditions du terrain. Étant donné que par de telles techniques, les boulons seront bien posés, il sera possible de réduire les coûts du reboulonnage des sections de galeries où le soutènement initial s'est révélé inadéquat ou insuffisant.

Il me serait impossible de nommer toutes les personnes, exploitants miniers, chercheurs et fabricants d'équipement, qui ont contribué à la réalisation de ce guide. Je les remercie toutes sincèrement pour leur contribution et leur aide. Sur le terrain, les travaux ont été réalisés avec l'aide et la collaboration des exploitants des mines Bousquet, Camflo, Corbet, Dest-or, Gaspé, Kiena, Lac Shortt, Montauban, Niobec et Opemiska. Je remercie tout particulièrement François Charette pour les travaux remarquables qu'il a effectués sur le terrain, Ivan E. Hugo, anciennement au service de la Eimco Secoma, pour sa contribution au chapitre sur le boulonnage mécanisé, ainsi que Jean-Marc Robert, directeur de division au CRM, et Louis Bienvenu, responsable des activités de contrôle du terrain au CRM, pour les conseils et le soutien qu'ils ont fournis tout au long du projet.

Enfin, j'espère que la version anglaise du guide suscitera autant de discussions et de réflexions que la version originale en français. La version anglaise est légèrement différente de la version française, car elle incorpore les développements technologiques les plus récents, ce qui en fait un guide tout à fait à jour. La dernière partie du chapitre 4, qui traite du choix de la longueur et de l'espacement des boulons, a été complètement révisée, et un chapitre sur le boulonnage mécanisé a été ajouté. La portée et l'esprit du guide sont toutefois demeurés intacts.

P. Choquet

Pierre Choquet

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Chapter 1: INTRODUCTION

This chapter defines the objectives and content of this guide and outlines its specific contribution in relation to other rock bolting utilization guides.

Chapter 1:
INTRODUCTION

Rock bolting is the most commonly used method to support mine galleries and is also widely used in mining stopes. The annual consumption for underground mines in Quebec has been estimated to be about 750,000 bolts in 1984 of which most (that is, 72%) are mechanically anchored bolts. Bolts grouted with resin or cement are being increasingly used, and account for about 18% of total consumption. Finally, friction bolts, such as Split Set and Swellex, both of which have been introduced relatively recently (since 1977 and 1982 respectively) are the third types of support bolts generally found in the mines, and they account for 10% of the total utilization figure.

Apart from the wide variety of bolts presently available on the market, there are several factors that contribute to the need to develop a Rock Bolting Practical Guide that is suitable for the particular mining exploitation conditions in Quebec, or more generally, of metallic and non-metallic mines other than coal mines. In particular, we could mention:

- the increase in the width of galleries, compatible with large Load-Haul-Dump (LHD) equipment;
- the increase in the depth of exploitation in several mines, which is often accompanied by an increase in the unit consumption of bolts;
- the greater, although still limited, use of bolting jumbos which, by distancing the miner from the ground, contribute to the increasing preference to use standardized bolting patterns, which are always denser than when bolting is used as a function of local needs;
- a general concern with the safety of miners in the workplace, which is accompanied by a certain devaluation of the traditional rock face scaling work, and tends to replace it by the use of the previously mentioned standardized bolting patterns, and an increased utilization of protective wire mesh in the roof of the galleries.

This guide is the result of a series of meetings, field studies, and reviews of the literature that were carried out to take stock of presently used bolting practices and the state of knowledge on this voluminous subject. One of its main objectives is to allow an informed choice of the type of bolt that is most suitable for the ground conditions encountered in a particular site, and subsequently to ensure its proper installation.

The guide follows a certain chronological order that suggests the way in which we should approach or, more often than not, re-examine mine support policies.

Thus, following this introductory chapter, where the objectives and specific contributions of the guide are discussed, Chapter 2 will describe briefly the mechanisms generally used to explain the effects of rock bolting. Even though the various phenomena discussed (convergence of the wall, formation of a natural vault) are rather difficult to observe in the day-to-day activities of the mine, many people will nevertheless find the chapter useful to explain the reasons why rock bolts are installed and their purpose.

Chapter 3 contains an exhaustive description of the four principal types of support bolts in present use. This chapter is altogether independent of the others, and will be of particular interest to those involved in supervising the installation of one or more of these types of bolts. It contains a description of their manufacturing characteristics, mechanical properties, holding power, accessories such as head plates, hardened steel or wooden washers, as well as the rules that must be followed in order to ensure their proper installation, such as the tolerance respecting the diameter of the drill holes. The types of ground for which each of these bolts is suitable are also mentioned. A reading of this chapter will make it possible to understand the advantages and disadvantages of using one type of bolt over another and could lead to the testing of bolts different from those actually used in a particular site, in view of their eventual large-scale utilization.

Chapter 4 deals with the selection of rock bolting characteristics (bolt length and spacing) as a function of ground conditions. This chapter is also independent of the preceding ones. Its objective is to review as well as contribute to the diffusion of ground exploration methods that are not widely used in practice because of the difficulties involved in their application. These methods remain unused in spite of having been recognized as being extremely useful by people involved in questions of support, particularly in the case of ground classification systems known as "geomechanics classifications." In some ways, this chapter can be seen as a tool to bring about a reconciliation with methods too often reserved for specialists alone. As part of the preparation of this guide, these same methods were tested in many Quebec mines, and this made it possible to describe methods that have been truly tested in the field, and for which the simplicity of installation was one of the prime criteria. Various points related to the selection of bolting characteristics in particular circumstances, as in the case of grounds where rock bursts are likely to occur, are also discussed in this chapter. Chapter 5 explains some principles of mechanization of rockbolting.

Finally, to meet the objective of the guide, which was to produce a practical work that could be consulted easily, the more detailed discussion of certain test results, calculation methods, and specific points that would otherwise needlessly burden the text are all included in the appendices.

1.1 ORIGINALITY IN RELATION TO OTHER GUIDES

Many rock bolting utilization guides that can be used in the geological context of Quebec mines have already been published, including the following:

- the CANMET guide (Energy, Mines and Resources Canada), 1970 (1,2)*;
- the guide published by the Norwegian Geotechnical Institute, 1976 (3);
- the guide published by the Mining Industry Society/Société de l'Industrie Minérale, 1980 (4);
- the guide of the Norwegian Institute of Rock Blasting Techniques, 1979 (5);
- the guide of the French Underground Mining Association/Association Française des Travaux en Souterrain, 1979 (6);
- the guide published by the U.S. Army Corps of Engineers, 1980 (7);
- the guide of the Construction Industry Research and Information Association, 1983 (8).
- *Professional Users Handbook For Rock Bolting* by Bengt Stillborg, 1986 (9).

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* The references cited are listed after Chapter 6.

However, we should point out that, with the exception of the first and the last guide listed above, and to a certain extent the third and fourth, all these guides are mainly concerned with the construction of tunnels and other permanent civil engineering installations. Moreover, they have generally been developed on the basis of the experience acquired in this particular context, even though admittedly several methods and types of equipment are also suitable for the mining sector.

Table 1 was prepared to provide a quick overview of the content of these guides. The various headings shown are those treated in this guide.

We should draw attention to a particular case: guides specially designed for rock bolting in coal mines. Because of the sedimentary nature of these deposits, which are very often accompanied by horizontally bedded gallery roofs, rock bolting in coal mines is a special case and has been studied many times, particularly in the United States by the U.S. Bureau of Mines. The following guides are noteworthy:

- the guide of the U.S. Bureau of Mines (studies carried out by L.A. Panek), 1973 (10);
- the guide of the U.S. Bureau of Mines (contract with the University of Kentucky), 1974 (11);
- the guide of the U.S. Bureau of Mines (contract with Michigan Technological University), 1980 (12);
- the guide published by Pennsylvania State University, 1984 (13, 14).

The originality of this guide in relation to those already published appears in many specific aspects as shown in Table 1. Here, in the following chapters, the emphasis is on the exhaustive description of the various types of support equipment available, including the friction bolts that have recently appeared on the market. The exploration of ground conditions also receives considerable attention, and the relationship between these and the various types of bolts and accessories used is particularly emphasized. The guide also contains a summary of the results of pull-out tests carried out on the various types of bolts, which will facilitate an understanding of their operating mechanisms. Finally, practical recommendations that make it possible to ensure proper installation and a follow-up of the various bolts used are included throughout the text.

	Effects of Rock Bolting	Exploration of Ground Conditions	Description of Various Types of Bolts and Accessories (Other than Split Set and Swellex)	Description of Split Set and Swellex Bolt Barrels	Selection of the Suitable Type of Bolt as a Function of Ground Conditions	Selection of Character-istics (Length. Spacing)	Selection of Accessories (Head Plates. Wire Mesh)	Recommendations for the Inspection and Follow-up of the Bolts Installed
CANMET		X				X	X	
NGI		X			X	X	X	
MIA	X		X		X	X	X	X
NIRBT	X	X	X		X		X	
FUMA		X			X	X		
USCE	X		X		X	X	X	X
CIRIA	X		X			X		
STILLBORG	X	X	X	X	X	X	X	X

Table 1 – Comparison of Items Discussed in the Main Rock Bolting Utilization Guides

1.2 PREPARATION OF THE GUIDE

This guide was prepared between 1983 and 1986 and involved the following tasks:

- May–August 1983 and May–August 1984: visits to 25 Quebec mining operations to collect information on their use of rock bolting. Two summary reports on these visits were produced (15, 16);
- May–August 1985: survey and measuring campaign regarding the use of rock bolting in 10 Quebec mines. The results are used within the framework of this Guide and have also been separately published (17, 18);
- January–December 1985: discussion of the content of the guide during the course of regular meetings of the Ground Control Committee of the Quebec Mining Association (QMA).
- January–December 1985: meetings with manufacturers of rock bolting equipment (Stelco, Stewart, Ingersoll–Rand, Atlas Copco), and missions to Ontario (Ministry of Labour of Ontario, operating mines) and the United States (U.S. Bureau of Mines: Research Centers in Spokane and Denver; Mines Safety and Health Administration: Denver regional office, operating mines). A report on these missions was published separately (19).

Chapter 2: THE EFFECTS OF ROCK BOLTING

The three principal effects of bolting are discussed. This discussion makes it possible to explain the support mechanisms in schistous or heavily jointed rocks, as well as in rocks that show a random fracture network. In the case of the latter, there are two opposing theories: does rock bolting contribute to the formation of a natural or of an artificial rock vault in the roof of the excavations? By answering this question, it is possible to determine whether or not it is necessary to use tensioned or non-tensioned rock bolts.

Chapter 2:
THE EFFECTS OF ROCK BOLTING

The effects of rock bolting can be divided into three large categories:

1. suspension;
2. the support of bedded or schistous grounds;
3. prevention of fissure movement in grounds that are fractured in several directions (keying effect). These three mechanisms are illustrated in Figure 1.

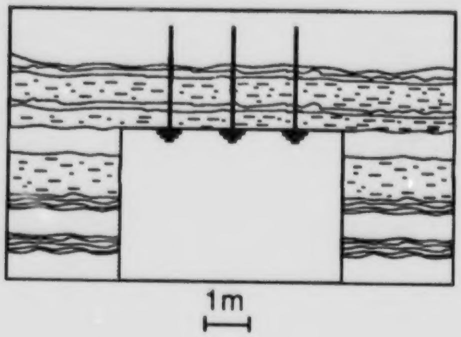
It is interesting to note that in the entire rock bolting literature, there seems to be general agreement regarding the existence of only these three discrete mechanisms, plus several variants derived from them. This does not preclude the existence of a wide variety of studies and rock bolting dimensioning methods in the literature. Consequently, as we will see below, even if the mechanism is known, the way in which it can be taken into account by calculation and design methods is not necessarily simple, particularly in the case of the third mechanism.

2.1 GROUND SUSPENSION

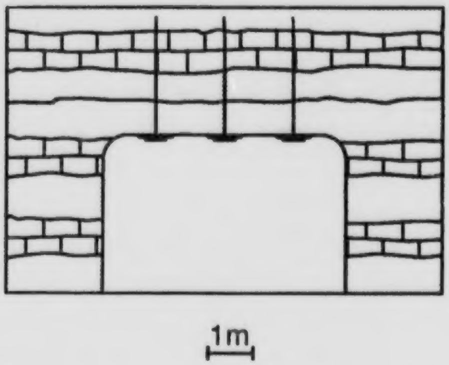
This effect is the simplest to design in the case of bolting. Thus, we presume that the support must carry the entire weight of a ground layer that is too thin or whose quality is too poor to allow the keying effect of the third mechanism to keep it in place.

In this case, the rock bolting system must be chosen in such a way that it is safely anchored in a ground layer of better quality. The holding power, quality and minimum grouting length required for each of the main types of bolts are discussed in Chapter 3.

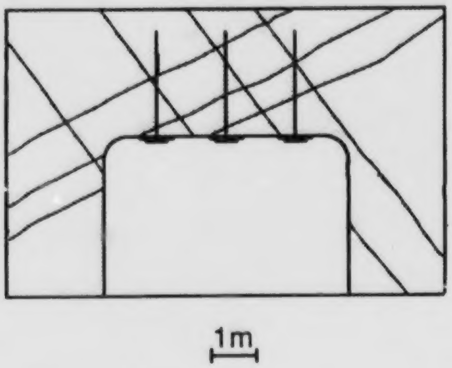
Moreover, the diameter and spacing of the bolts must be such that the bolts can safely bear the total weight of the ground layer while also taking into account a sufficiently high safety factor (1.25 to 1.75).



a) Suspension effect



b) Effect of reinforcing bedded grounds



c) Effect of preventing the movement of fissures in fractured grounds (keying effect)

Fig. 1 - Three effects of bolting in underground galleries

The suspension effect applies only in extreme situations encountered in certain sedimentary grounds where the stratification is relatively horizontal. In this case, we generally choose bolts with a high ground-holding power, for example, resin-grouted bolts or Swellex friction bolts. An example of the situation where we must consider suspension as the effect of bolting is that reported by Wijk and Skogberg (20) in a mine where the mineral deposit is overhung by a limestone layer 0.5 m thick, followed by a very fissured rock layer 1.5 m thick, before finding again a thick limestone layer with good holding power. This situation is illustrated in Figure 2 below.

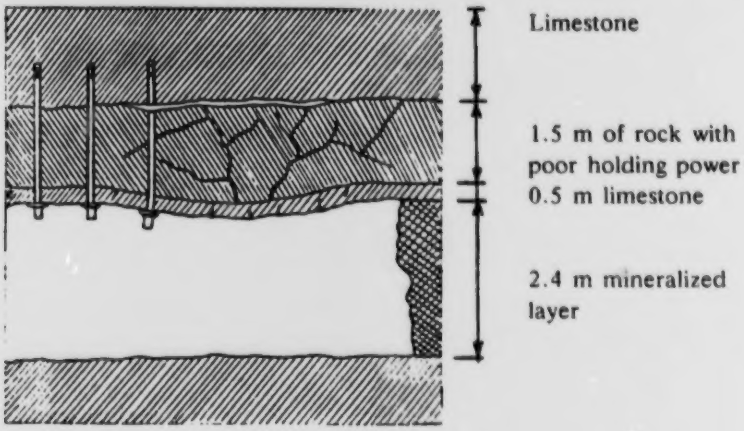
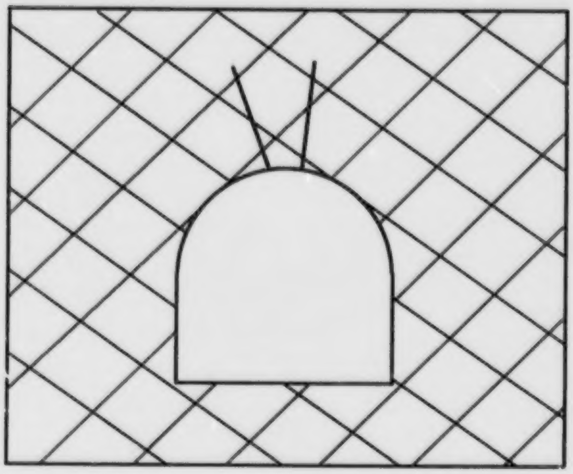
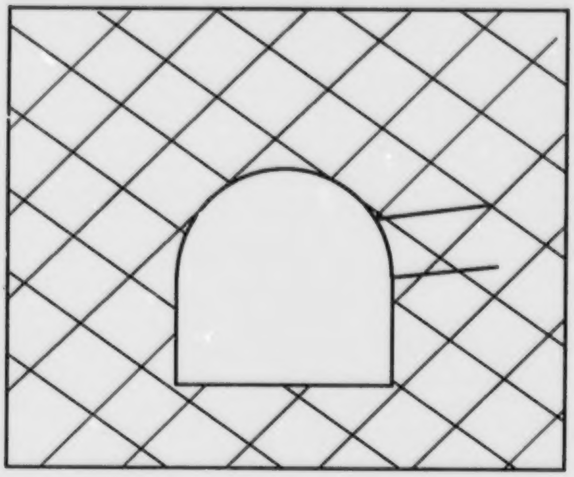


Fig. 2 – Bolting to suspend the ground in a sedimentary mine. From Wijk and Skogberg (20)

An important variant of the suspension effect is the case where bolting is called upon to support an isolated block cut off by large fissures in the roof or walls of an underground excavation (Figure 3). In this case, the dimensioning method consists of evaluating the weight of the block that is in the process of falling or sliding, and then to choose the diameter, length, and spacing of the bolts to ensure that the block will remain in place. A graphic method that can be used to evaluate the volume of a block cut by three fissures in the roof or walls of an underground excavation is described by Hoek and Brown (21, p. 185–191). When the block is in the process of sliding rather than falling, the formula shown in Appendix 1 makes it possible to evaluate how much bolt tension is necessary to stabilize it.



a) Suspension in the roof of the gallery



b) Increasing the sliding resistance of a
block on the wall of the gallery

Fig. 3 - Bolting used to suspend an isolated block

2.2 REINFORCING BEDDED GROUNDS

This bolting effect receives special attention in coal mines located in sedimentary ground, where the roof of the galleries is always stratified to a certain extent. On a worldwide scale, probably most bolts are installed in this type of situation. For example, in the United States alone, 120 million bolts are installed every year in coal mines.

In this case, the effect of bolting is relatively easy to understand; its function is to stabilize the first strata of the roof by friction to form a thicker and self-supporting beam. It is important to point out that the stabilized layers are subjected to forces represented on the one hand by their own weight, and on the other by horizontal stresses produced by the concentration of natural stresses in the wall that tend to make the layers sag toward the inside of the gallery (Figure 4). Thus, these horizontal stresses, whose intensity varies as a function of various parameters such as the shape and depth of the gallery, play a role in increasing the negative effects of gravity. The origin of the horizontal stresses, or more precisely, those that are parallel to the wall, can be easily explained by the theory of elasticity and is discussed in all general handbooks dealing with rock mechanics. Appendices 3 and 4 contain a summary of the distribution curve of the stresses in the walls of underground openings.

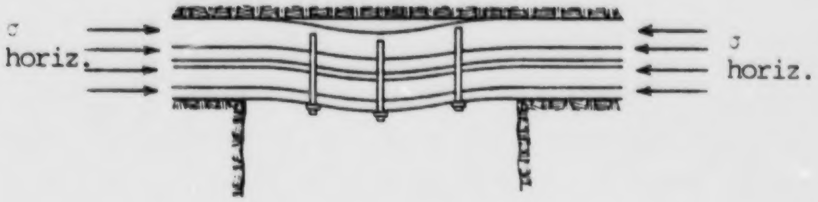


Fig. 4 – Sagging of layers under the combined effect of gravity and horizontal stresses

Far-reaching studies have been and continue to be undertaken in the United States, particularly by the U.S. Bureau of Mines, to arrive at a completely rational rock bolting dimensioning system for horizontally bedded grounds. Because of the relatively regular nature of these grounds, the results of forecasting calculations can probably be verified with a certain degree of accuracy.

The main studies undertaken within this framework are those previously mentioned in Section 1.1. Of these, the best known (although the oldest) are the studies carried out by Panek (9). Among other things, he has produced a graph for dimensioning tensioned bolts (see Appendix 2), which is still widely used.

A case of bolting bedded grounds that is of interest in mines opened in igneous rock, is the bolting of schistous grounds where the schistosity is parallel to the wall of the galleries or stopes, as shown in Figure 5 below.

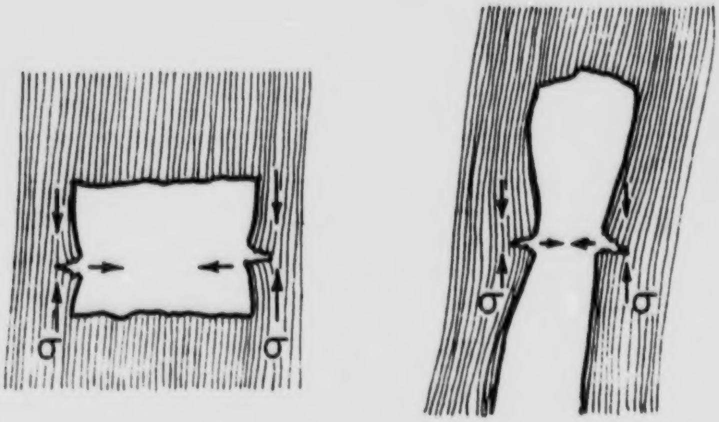


Fig. 5 – Buckling of layers in the walls of a gallery and stope in schistous ground

The high stresses that are parallel to the walls are still active in this configuration, but this time they tend to make the layers of the wall buckle toward the inside of the gallery.

Appendix 5 describes a simple solution based on the beam bending theory that makes it possible to evaluate the bolting system required to stabilize the buckling of schistous layers in the wall of a gallery or a stope.

2.3 PREVENTING THE MOVEMENT OF FISSURES IN JOINTED GROUNDS

This mechanism is commonly called upon to explain the remarkably successful effect of bolting on the stabilization of the roof of underground excavations in heavily jointed rock masses. Such grounds are typical of metallic mineral mines that are often located in magmatic (granite, diorite, gabbro) or volcanic (rhyolite, andesite, basalt) igneous rocks, or in metamorphic rocks (schists and granitic gneiss) of Precambrian or very ancient Cambrian origin.

In Quebec, the mines located in the Canadian Shield are very similar to this geological type, although those located in very schistous volcanic formations (tuffs, sericitic schists, etc.) may also be subject to the buckling of layers mentioned in Section 2.2.

The places where we are most likely to find a stress-free zone (see Appendix 4) of a certain thickness around the boundary of underground openings are highly jointed grounds where the joints are randomly oriented or organized into several sets of common orientation. In this stress-free zone, movement along the fissures is not restricted by stresses which, when they act perpendicularly to the plane of the fissures, help to keep them closed and minimize the effects of gravity.

In this situation, the real effect of bolting is not as direct or easy to understand as it is in cases where the bedded grounds are suspended or reinforced. The best explanation of the phenomena involved could be that of Bergman and Bjurström (22), which is discussed below, even though these authors are primarily ardent defenders of the increased use of non-tensioned bolts and bars, and consider that the artificial vault created in the roof of the openings by tensioned bolts has only a very small effect. So there are two schools of thought on the effect of bolting in fissured grounds, and it is possible to distinguish between partisans of the formation of a natural rock vault, like Bergman and Bjurström, and partisans of the creation of an artificial vault by bolting. This second approach, which is embraced by several authors, such as Lang (23), will be reviewed below.

2.3.1 The Mechanism of Forming a Natural Rock Vault in the Roof of Underground Openings

The effect of bolting in jointed ground as seen by Bergman and Bjurström is formulated to take into account that non-tensioned bolts are widely used in Sweden, and generally used in Europe, in situations where tensioned bolts would otherwise be used.

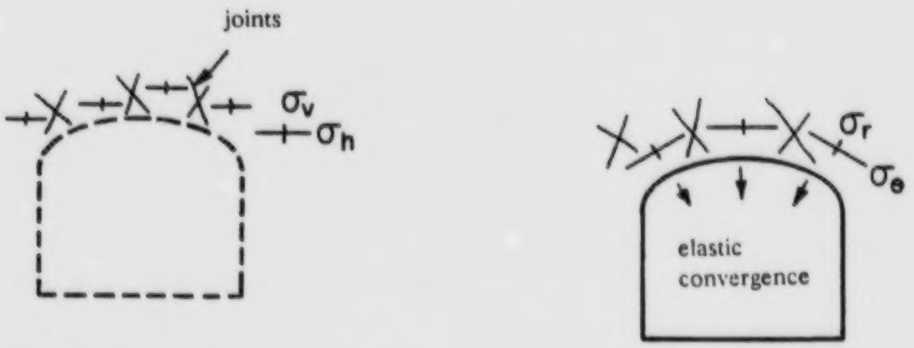
Non-tensioned bolts are generally grouted with cement or resin; however, the authors view the rapid success of the recently introduced Split Set and Swellex friction bolts as further proof of their theory.

It is now accepted, and many field measures tend to confirm it, that when a cavity is opened, a certain amount of convergence, that is, a narrowing of the space between the back and the floor, is produced. This convergence can be divided into two parts: an elastic part that takes place quickly, as soon as the cavity has advanced by a distance more or less equal to its diameter; and a delayed part that manifests itself when the advance is three to four times the diameter.

This delayed part is the one that interests us because it corresponds to the formation of a natural vault by the rock mass in the stress-free zone that subsides under the effects of gravity. This convergence gives rise to compressive stresses that, even though relatively weak in comparison with those that prevail before the cavity is opened, are nevertheless enough to keep the joints in the rock mass closed. These stages are shown in Figure 6.

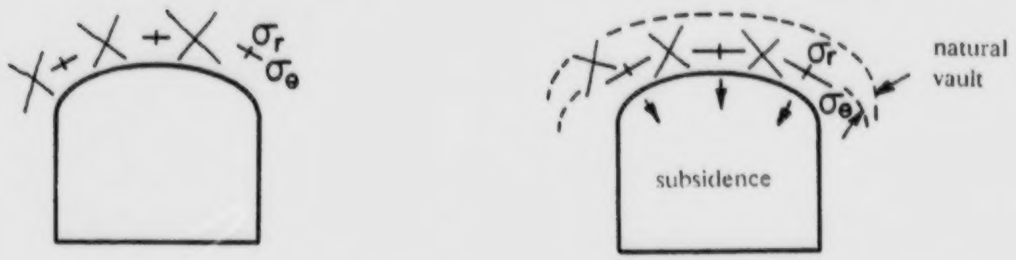
Under these conditions, the effect of bolting is very superficial. Its only effect is to keep in place the roof blocks to prevent slumps that would lead to the loosening of the grounds and accompany the subsidence of the rock mass while the natural vault is constructed. Thus, the bolts should not be longer than the thickness of the natural vault that is in the process of formation, to prevent the bolts from being needlessly tensioned and changing the regular distribution of stresses in the vault. It would be preferable to install several shorter bolts rather than a few long bolts that would be anchored mainly outside the vault.

For a theoretical analysis that would make it possible to evaluate the height of the natural vault and subsequently the required bolt length, see Reference 12, Vol. 4, p. 29-38 and Vol. 5, p. 86-92. However, in this guide, we suggest that the required bolt length be determined by the methods described in Section 4.4.


joints
σ_v
σ_h
σ_r
σ_θ
elastic convergence

a) Before opening the cavity: natural stress condition: σ_v and $\sigma_h = k_o \cdot \sigma_v$ ($k_o \approx 2$). The joints are closed by compression(σ_v and σ_h are vertical and horizontal initial stress fields).

b) Opening of the cavity, advance about equal to the cavity diameter: stresses concentrated on the wall, σ_θ = 3 *to* 5 $\sigma_v, \sigma_r = 0$. Joints perpendicular to stresses σ_θ are kept closed. Elastic convergence of the walls of a few millimetres(σ_r and σ_θ radial and tangential stresses to the cavity boundary).

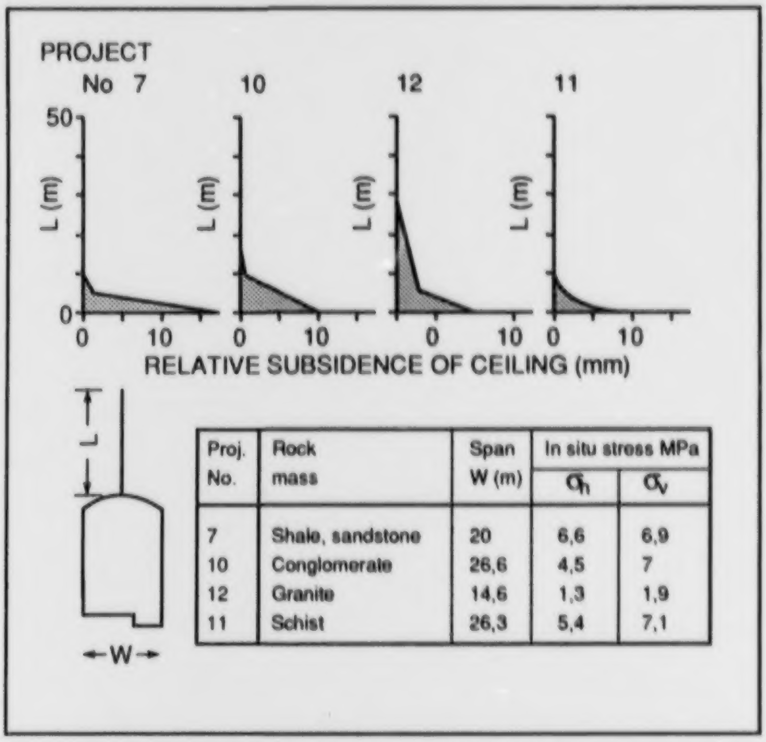

σ_r
σ_θ
natural vault
subsidence

c) Advance in the order of3 to 4 times the cavity diameter: creation of the stress-free zone (Appendix 4), $\sigma_\theta < \sigma_v, \sigma_r = o$. The joints are decompressed. Initiation of subsidence.

d) Subsidence under the effect of gravity (about 10 mm or more). Creation of a natural rock vault. Stresses are partially restored: joints are recompressed.

Fig. 6 - Mechanism of the formation of a natural rock vault in roof of underground cavities

In support of the formation of a natural rock vault in the roof of underground cavities, Bergman and Bjurström cite the results obtained by Hibino *et al.* (24) on measures of internal expansion in the rock mass in the roof of 12 underground chambers between 14 m and 26 m wide, located at shallow or medium depths, in Japan. The results of these measures are shown in Figure 7, where we can see that the greatest part of the expansion is produced in the first 5 m of the roof. The break in the curves indicates the probable boundary of the first natural vault. A little expansion is then visible to a depth of 10 m, where ground subsidence is then completely stabilized.



Proj. No.	Rock mass	Span W (m)	In situ stress MPa	
			σ_h	σ_v
7	Shale, sandstone	20	6,6	6,9
10	Conglomerate	26,6	4,5	7
12	Granite	14,6	1,3	1,9
11	Schist	26,3	5,4	7,1

Fig. 7 – Internal expansion in the roof of underground chambers. From Hibino *et al.* (24)

Concerning rectangular galleries, which are very common in many mines, it is reasonable to expect that a natural vault also forms in the stress-free zone of the roof through a mechanism similar to that described in Figure 6. However, since a rectangular shape is less favorable than a rounded shape for the roof, it is probable that the formation of the vault takes place at a certain depth in the roof, which leaves a greater volume of rock to be kept in place by bolting to prevent the initiation of a loosening of the ground, as shown in the diagram below.

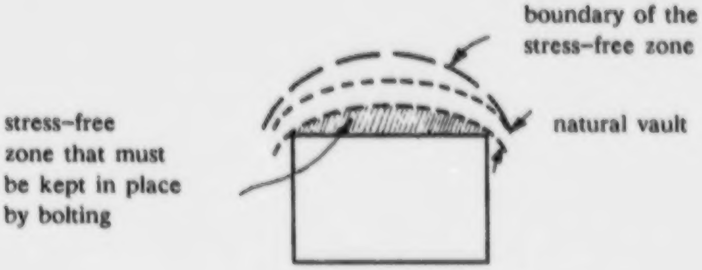

boundary of the stress-free zone
stress-free zone that must be kept in place by bolting
natural vault

Fig. 8 - Natural vault in a rectangular gallery

2.3.2 The Mechanism of Forming an Artificial Rock Vault Using Tensioned Bolts

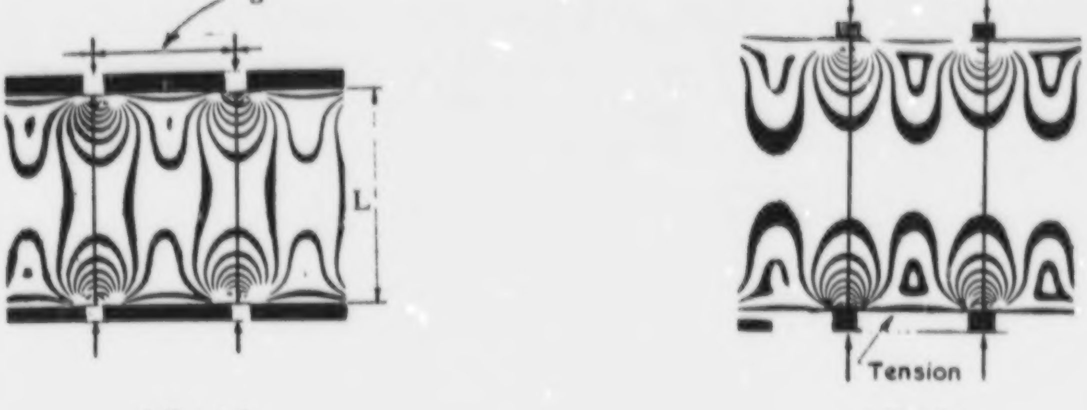
This bolting effect was introduced very quickly by the first users of rock bolting. These users were aware of the presence of a stress-free zone in the roof of underground excavations. It was natural for them to think that the installation of tensioned bolts in this zone would contribute to the creation of an artificial vault. This conclusion was based on experimental results like those obtained by Lang (23). Figure 9 shows one of his experiments: photoelastic models of a beam crossed by tensioned bolts, with various length/spacing ratios.

Two conclusions could be derived from these figures and were subsequently widely disseminated. The first is that the L/S ratio must be equal to 2 or higher, in order for uniform compression zone to develop between the ends of the bolts. The second conclusion is that a tension zone develops between the heads of the bolts, and that this may facilitate the crumbling of the rock in this location. To minimize the extent of this zone, it is preferable, as in the previous case, to ensure that the L/S ratio is at least equal to 2.

With the help of these experimental results, we can expect that the artificial rock vault will take the shapes shown in Figure 10, depending on the length of the bolts and the chosen L/s ratios. The figure illustrates the case of a gallery 5 m wide with two lengths of bolt L = 1.5 m, and L = 2.1 m, as are often encountered in practice.

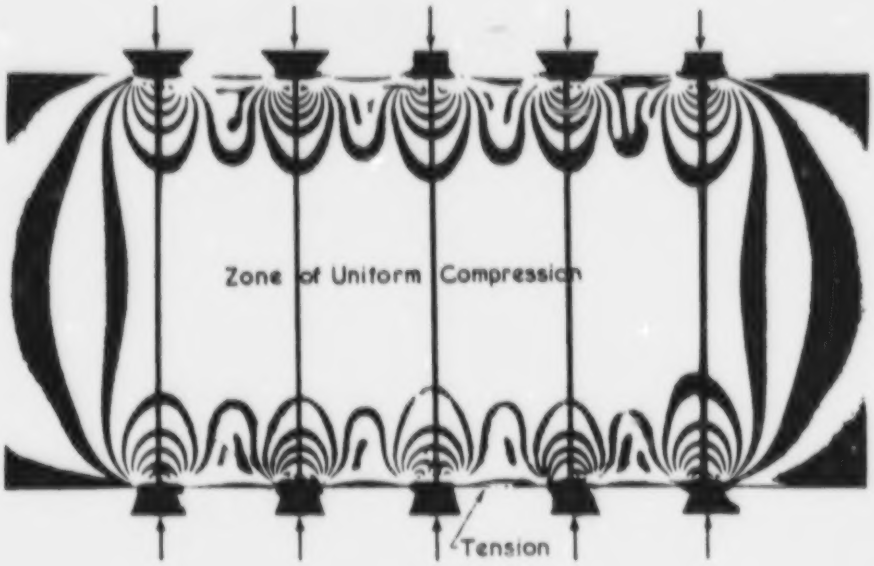
As we can see, when we choose an L/S ratio of 1.3, the compression zones of each bolt do not join together, and a significant tension zone is created between the bolts. Unfortunately, this L/S ratio of 1.3 is often encountered in practice, particularly when bolts 1.5 m (5 ft) long are used. This leads to the conclusion that, under these conditions, the bolts do not fulfill their expected role the creation of an artificial vault. If we want to keep a spacing of 1.2 m (4 ft), as is often found in practice, it is better to use bolts longer than 2.1 m (7 ft), since this situation is equivalent to the last diagram in Figure 10 where the artificial vault is very thick.

Under practical bolt utilization conditions, we could expect that the artificial vault created would not be as perfect and regular as those shown in Figure 10. When the bolts are of large diameter and carefully tensioned up to a significant fraction of their yield strength, an artificial vault may be created; however, this is much less likely in galleries where mechanically anchored bolts are only slightly tensioned, and are even often loosened as the result of blast vibrations or the crumbling of the rock near the heads. It is much more reasonable to consider that the function of these slightly tensioned bolts is rather to participate in the formation of a natural rock vault as discussed in Section 2.3.1.



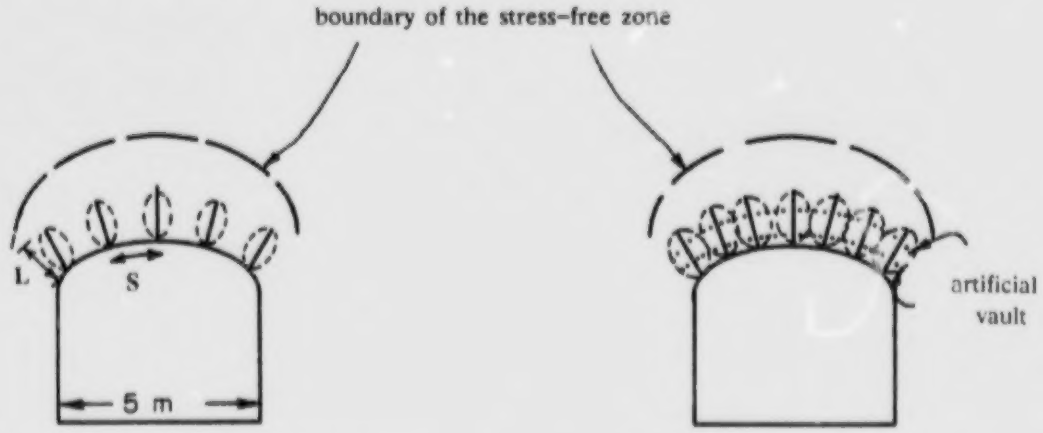
L/S = 1.5

L/S = 2.0


Zone of Uniform Compression
Tension

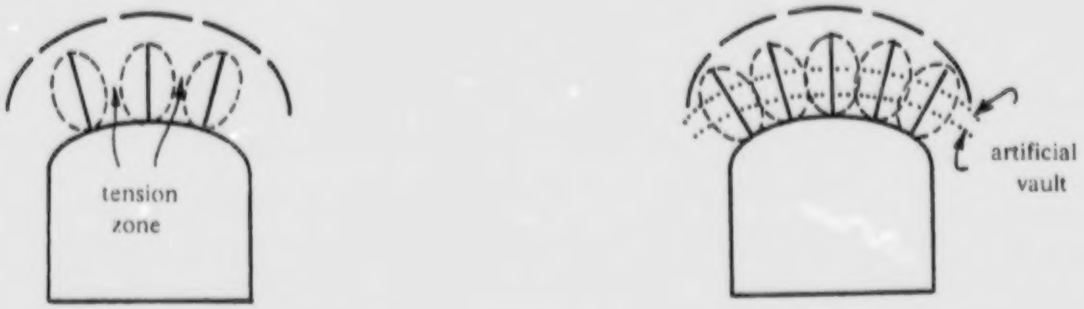
L/S = 3.0

Fig. 9 – Photoelastic models of the effect of tensioned bolts. From Lang (23)
(L: length of the bolts)
(S: spacing of the bolts)


boundary of the stress-free zone
L
S
5 m
artificial vault

L/S = 1.3 (L = 1.5 m, S = 1.15 m)
No artificial vault is created.
Large tension area between the bolts.

L/S = 2 (L = 1.5 m, S = 0.75 m)
Creation of an artificial vault.
Reduced tension zone between the bolts.


tension zone
artificial vault

L/S = 1.3 (L = 2.1 m, S = 1.6 m)
No artificial vault is created.
Very large tension zone between the bolts.

L/S = 2 (L = 2.1 m, S = 1.05 m)
Creation of an artificial vault.
Tension zone between the bolts.

Fig. 10 - Various ways an artificial rock vault can be created by the use of tensioned bolts, depending on the length and spacing of the bolts

Chapter 3: DESCRIPTION AND PERFORMANCE OF VARIOUS TYPES OF BOLTS

The four principal types of bolts are discussed. The following information is given for each of them:

- mechanical properties (load at rupture, etc.);
- description of principal accessories (head plates, resins, wire mesh, etc.), as well as the advantages and disadvantages of their utilization;
- pull-out force and principal parameters that affect it (diameter of the drill hole, composition of the cement grout, mixing time of resin cartridges, etc.);
- particular problems of each type of bolt (loss of tension in mechanically anchored bolts, excessive sensitivity of the bolt to the diameter of the drill hole, etc.).

Chapter 3:
DESCRIPTION AND PERFORMANCE OF VARIOUS TYPES OF BOLTS

3.1 MECHANICALLY ANCHORED BOLTS

As previously mentioned, this type of bolt is the one most commonly used in Quebec mines. It consists of a steel bolt with one threaded end to which an expansion shell can be fitted. The other end is a forged head or it can also be threaded to fit a nut. These bolts are always used with a head plate placed near the head, because they have been designed to be tensioned by tightening them mechanically.

In Quebec, the main manufacturers of mechanically anchored bolts are Stelco* (Montreal) and Produits Miniers Stewart (Noranda). From Montreal, Stelco serves all of the Canadian market and also sells to the United States. Northway Metals (Sudbury), another Canadian company, also manufactures mechanically anchored bolts.

Apart from Stelco, which manufactures its own bolt barrels, the two other companies buy the bolt barrels, often from Stelco. These bolt barrels are cut in the workshop to the desired length, and then the Universal National Coarse (UNC) threads at one or both ends are cold forged by rotating the bolts between dies rather than machining them, so that the threads are not a weak point when the bolt is put under very high tensions. The head is finally hot hammered in about three passes, to preserve the internal structure of the steel, particularly at the point of contact of the head and the bolt (25). The lubricant used when forging the threads is not removed; it acts as a rust protector and helps improve the ratio between the tension and the installed load torque. Finally, the expansion shell is added to the bolt. Generally, these expansion shells are produced by a different manufacturer, the largest of which is Frazer and Jones (Syracuse, N.Y.). They make many anchors of various models and patents that will be discussed below.

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*In 1987, Ground Control from Sudbury purchased Stelco's rock bolt manufacturing plant and moved it to Sudbury.

To end, we would like to mention that there is an American Society for Testing and Materials (ASTM) standard that regulates the manufacture of rock bolts in the United States (26). This is ASTM Standard F-432-83: Standard specification for roof and rock bolts and accessories. Even though it is not mandatory in Canada, Canadian manufacturers comply with many of its provisions.

3.1.1 Properties of the Bolt Barrels

The bolt barrels are available in lengths from 45 cm (18") to 6 m (20 ft), and diameters, from 15.9 mm (5/8") to 34.9 mm (1 3/8"). Currently, the most common diameter in Quebec is 5/8", and less frequently 3/4".

The steel used is generally high strength steel (690 MPa). In the case of bolt barrels manufactured by Stelco, the mechanical properties are as follows (27, 28):

Yield strength: 60,000 psi (414 MPa)
Ultimate strength: 100,000 psi (690 MPa)

Load exceeding the yield strength (diam. 5/8"): 13,560 lb (60.3 kN)

Load exceeding the yield strength (diam. 3/4"): 24,040 lb (106.8 kN)

Load at rupture (diam. 5/8"): 22,600 lb (100.4 kN)

Load at rupture (diam. 3/4"): 33,400 lb (148.4 kN)

Percentage of elongation at rupture: 13% over 200 mm

It is important to remember that the mechanical properties listed by the manufacturers are always minimum properties. In practice, most units exceed the minimum values. Thus, during bolt pull-out tests carried out in the field, we frequently find that the bolt head gives way at 124.4 kN (28,000 lb), while its guaranteed load at rupture, as mentioned above, is 100.4 kN (22,600 lb) (Appendix 11).

Manufacturing workshops generally have a high degree of versatility because of the equipment they have available. Also, it is always possible to request particular modifications in terms of the manufacture of the bolt barrels. Some examples of these changes are listed below.

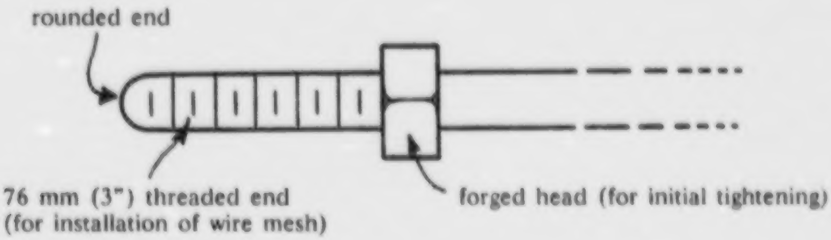
a) The threads are generally 140 mm (5 1/2") long, which is enough to ensure that the nut or threaded wedge of the expansion shell does not get caught on the bolt barrel during tightening. When there is doubt that the shell can remain clear of bolt barrel, as in the case of bolts used to install and support wire meshes where the operator often has a tendency to start to tighten before the mesh is applied to the wall, we suggest that the length of the threaded section be increased slightly.

b) Even though the barrels are generally threaded for tightening to the left, it is also possible to thread the bolt barrels to the right, in such a way that the tightening can be done only by using a power wrench rather than a stoper drill.

c) The free ends of double-threaded (DT) bolt barrels may be rounded to prevent the deterioration of the threads during blasting, and to facilitate the installation of a second nut that is used to support the wire mesh.

d) The length of forged head (FH) bolts may be stamped on the head.

e) Other particular modifications may be requested, such as the following design for a bolt head. This can be readily produced in most workshops, and has recently been requested for field testing by a mine in Manitoba.


rounded end
76 mm (3") threaded end
(for installation of wire mesh)
forged head (for initial tightening)

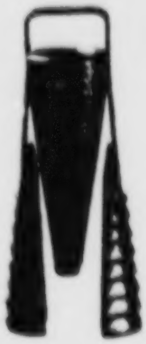
3.1.2 Description of Expansion Shells

There are many different expansion shell models that are suitable for a particular diameter of bolt barrel or drill hole, or for particular ground conditions (soft or hard rock, general use).

To avoid having to provide an exhaustive description, we will generalize by grouping the expansion shells made by Frazer and Jones into four main categories, and emphasizing that there is also a South African expansion shell made by Northway Metals that is widely used in Ontario mines.

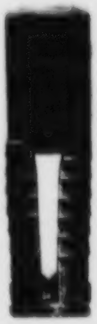
The five categories of expansion shells are: D, F, R, OB and TT6, the South African model. Most of these expansion shells are made in various models and sizes; however, since most drill holes used to install rock bolts in Canadian mines have a diameter of 31.75 mm (1 1/4"), it is interesting to note that, for all practical purposes, only a single model of each type can be found in circulation in the mines.

a) Type D Expansion Shells



This is a bail shell with two segments. The wedge has an elongated shape to ensure that the contact surface between the segments and the walls of the hole is as large and even as possible. The most commonly used model is the D 1 1/4 for drill holes with a diameter of 31.75 mm (1 1/4"). Its characteristics, as well as those of other expansion shells, are described later in the text in a summary table.

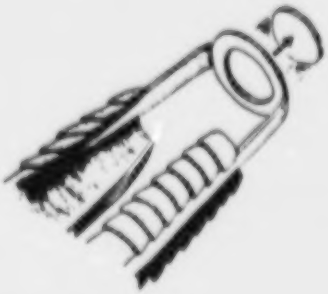
b) Type F Expansion Shells



This is a bail shell with four segments. The length of the wedge is shorter than that of the D shells, and this may reduce the uniformity of the pressure distribution along the segments. On the other hand, the four segments make it better suited to an irregular geometry of the drill hole.

The FS3BL shell is widely used in Ontario because its design, especially the shape of the wedge, allows it to be used in drill holes with a diameter of 31.75 mm (1 1/4") to 34.9 mm (1 3/8"). Given that the tolerance in the diameter of the drill holes for other types of shells is very small, +1.6 mm (+1/16") and −0.8 mm (−1/32"), this shell also makes it possible to retain better pull-out strength characteristics in most holes drilled with a 31.75 mm (1 1/4") bit, but for which overboring is often greater than the above-mentioned tolerances.

The bail of the FS3BL anchor can be equipped with a safety device called a "pop-out," as shown in the opposite illustration. This consists of a disk that is ejected if too much of the length is tightened and the barrel comes in contact with the bail, which would push the segments unevenly into the drill hole and reduce the holding power of the bolt. This device is mainly used with DT bolts to install the wire mesh in Ontario mines.

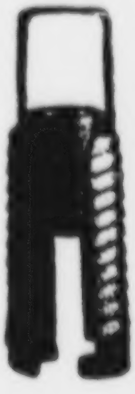


c) Type R Expansion Shells

The design of this type of shell, also called a "support shell", is older than that of the other bail expansion shells. Its four segments are attached at the base, and even if the theoretical contact surface between these and the wall of the hole is relatively long (the entire forged section), tests and x-rays (29) of drill holes have shown that the effective contact zone is smaller and is close to the wedge. At present, it is generally accepted that this type of shell is best used with very hard rocks in which it can have a high holding power. It is not very widely used, however, at this time.

d) OB Expansion Shells

This is a bail shell in four segments and is similar in design to the Type F shells.



e) TT6 Expansion Shells

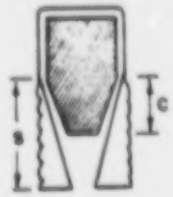
This also is a bail shell and has three segments. It is similar to the D 1 1/4 shell but has a much shorter wedge.

The table below summarizes the properties of the main expansion shells described above.

Expansion shell	Number of segments	Diameter of the bolt barrels (in)	Diameter of the drill holes (in)	Length of the segments* (in)	Length of the wedge** (in)
D 1 1/4	2	5/8 - 3/4	1 1/4	1.8	2.3
FS3BL	4	5/8	1 1/4-1 3/8	1.8	1.1
R	4	5/8 - 3/4	1 1/4	1.2	1.2
OB 1 1/4	4	5/8 - 3/4	1 1/4	2.7	1.4
TT6	3	5/8	1 1/4	1.9	1.2

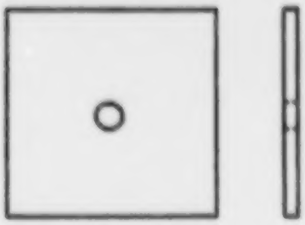
* Corresponds to the theoretical length of the contact surface between the segments and the wall of the drill hole(S).

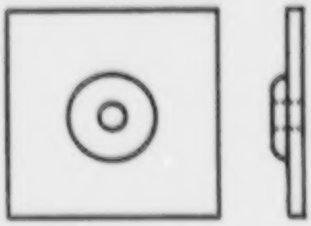
** Length of the part over which the segments slide when the shell expands(C).



3.1.3 Head Plates

It is necessary to use a head plate with mechanically anchored bolts. There are two principal types of head plates: flat plates and domed plates as shown below.

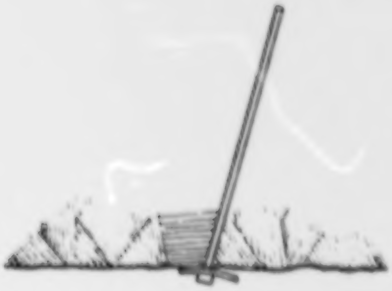




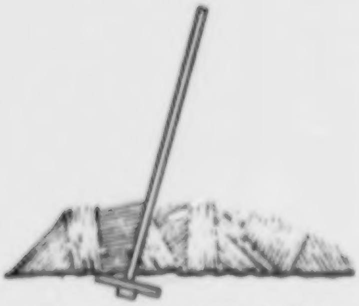
These plates are available in various sizes from 102 x 102 mm (4" x 4") to 203 x 203 mm (8" x 8"); the most common sizes are 102 x 102 mm (4" x 4") and 127 x 127 mm (5" x 5"). Their thickness may vary between 6.3 mm (1/4") and 9.5 mm (3/8").

The main purpose of using head plates is to contribute to the long-term maintenance of tension in bolts; thus, the selection of their characteristics must be made with this purpose in mind. The following are some observations that may be made regarding the performance of these plates:

- Thin plates 6.3 mm (1/4") generally bend when the bolts are tightened. This is a favorable characteristic, because most bolts are not installed on a completely flat surface and are often installed at a slight angle to the surface. The 9.5 mm (3/8") plates, particularly those measuring 102 x 102 mm (4" x 4"), are often stiff enough to prevent bending under the same conditions. This leads to the situation shown in the following illustration, which is not recommended, because the bolt will quickly lose tension.



Thin plate, 6.3mm (1/4"):
better contact surface



Thick plate, 9.5mm (3/8"):
does not bend

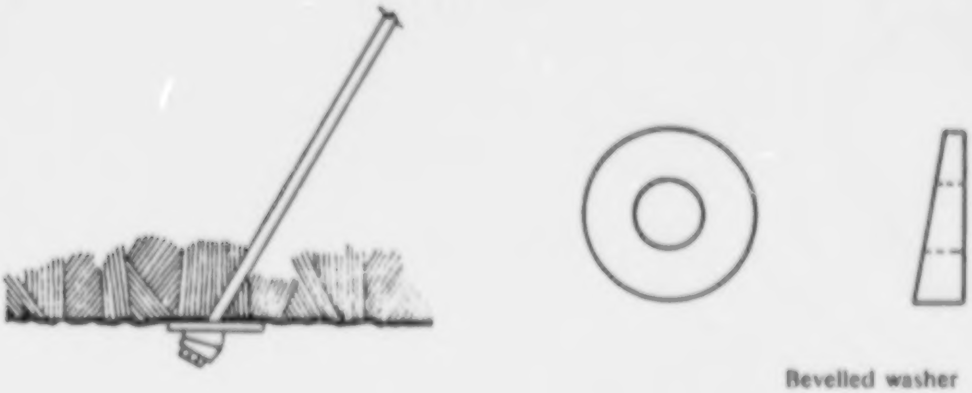
- In practice, the observation made above means that the plates must be chosen in such a way that, after tightening, and depending upon the method used (stoper hammer, power wrench, etc.), the plate must be in close enough contact with the wall. Other parameters that may play a role in the choice are resistance to corrosion in the presence of very acidic water (choose the thickest plates: 9.5mm (3/8")), the need to protect the wire mesh during tightening when bolts are used to support the mesh (use the largest plates: 127 x 127 mm (5" x 5") or 152 x 152 mm (6" x 6")), or the need to offer a larger contact surface with the wall of the gallery, when the rock and the collar of the hole tend to crumble (use 152 x 152 x 9.5 mm (6" x 6" x 3/8") plates).

- Various types of head plates that can be used to keep tension in the bolts have been designed and are available in the market. Domed plates of rectangular or triangular shape are often found. The latter, which are used in many American hard rock mines, rest only on their three points.

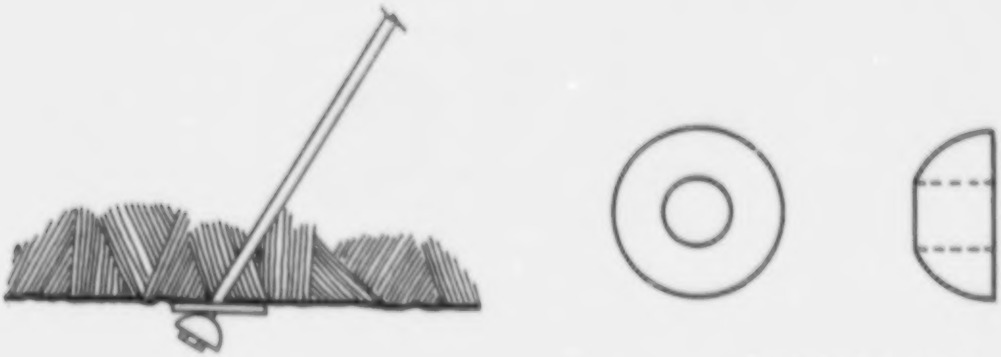
However, some test results have shown that the long-term loss of tension is lower with flat than with domed plates. This apparently surprising result occurs because domed plates remain susceptible to all the small displacements that can occur during the whole life span of the bolt; flat plates, if they are properly installed against the wall, are less sensitive to these displacements. Various studies confirming these results, as well as the performance of plates used with plywood and hardened steel washers, are discussed in Appendix 6.

Consequently, aside from particular circumstances, the use of flat plates is preferable to the use of domed plates. One exception to this rule is the installation of bolts at an inclination of more than 10°, where it is preferable to use a domed plate with an oval-shaped hole, and to tighten the bolt some time after installation to recover lost tension.

- Some mines advocate the use of bevelled washers (two washers used at the same time) or semi-spherical washers like those shown below, when the bolts are installed at an angle in relation to the wall.



Bevelled washer

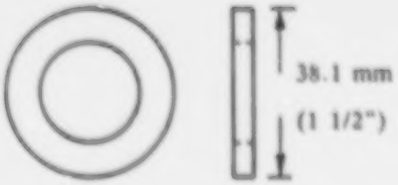


Semispherical washer

These practices are highly recommended. However, there are no experimental results to confirm that these washers do not cause long-term loss of tension in the bolts through a mechanism similar to that for the domed plates.

3.1.4 Hardened Steel Washers

It is possible to use a hardened steel washer (HSW) between the head of the bolt and the head plate. Its greater hardness helps reduce friction during tightening. For the same applied load torque, the tension in the bolt will increase, considerably in some cases.



Appendix 7 discusses the results of three series of bolt tightening tests with and without HSWs. The main conclusions that can be derived from these tests are:

- The use of HSWs significantly increases the tension in DT (double threaded) bolts (an increase of 80%), and to a lesser extent in FH (forged head) bolts (the increase may reach 40%). Using HSWs makes it possible to obtain tension from 4 to 5 tonnes (8,800 to 11,000 lb), for an applied load torque of 229 N-m (170 lb-ft).
- The use of HSWs is particularly recommended for FH and DT bolts when the bolts are used with a plywood washer and a wire mesh. Without HSWs, the level of tension in these two types of bolts is too low (1.3 to 1.55 tonnes: 2860 to 3410 lb), even with an applied load torque of 229 N-m (170 lb-ft).
- The use of HSWs is not recommended for FH bolts used with a head plate alone; using HSWs may cause shearing of the bolt head when the bolt is installed at an angle in relation to the wall. Moreover, tests have shown that, without HSWs but with an applied load torque of 229 N-m (170 lb-ft), these bolts already reach a level of tension of 3.6 tonnes (7,900 lb). The advantage of using HSWs in this case, in spite of the risk mentioned above, would be to reduce the scatter of tension values around the average value of 3.6 tonnes (7,900 lb).
- The use of HSWs can make it quicker and easier to install bolts, when tightening them to the required level of tension.
- The use of HSWs does not cause long-term loss of tension in bolts (see Appendix 6).

3.1.5 Wood Washers, Wood Blocks

In many mines, a wood washer is often used during the installation of the wire mesh. This washer is placed between the head plate and the mesh, and serves to protect the latter during the bolt-tightening operation, when the plate may shear the wires, which are often of small diameter (no. 8: 4.11 mm (0.162") or no. 9: 3.76 mm (0.148") gauge wire).

The wooden washer used is generally a piece of plywood measuring 152 x 152 x 12.7 mm (6" x 6" x 1/2"), with a head plate measuring 102 x 102 x 6.4 mm (4" x 4" x 1/4"). However, the experience of some mines that use no. 8 gauge 102 x 102 mm (4" x 4") wire mesh has shown that it is possible to replace the plywood washer with a larger steel head plate 127 x 127 x 6.4 mm (5" x 5" x 1/4"), and that the wire mesh is equally protected.

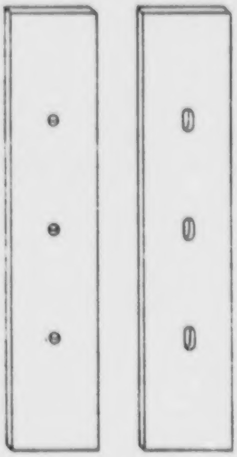
Appendices 6 and 7, which deal with the long-term loss of tension in the bolts and the use of hardened steel washers, also list the results of tests carried out on plywood washers. The main conclusions of these tests are the following:

- For an identical installed load torque of 229 N-m (170 lb-ft), the use of a plywood washer on the wire mesh causes a reduction of the initial tension in comparison with that of a bolt without wire mesh or plywood washer. The tests described in Appendix 6 indicate a reduction from 4.9 tonnes (10,780 lb) to 3.5 tonnes (7,700 lb) for bolts tightened as hard as possible with the stoper hammer, and those discussed in Appendix 7 indicate a reduction from 3.6 tonnes (7,920 lb) to 1.55 tonnes (3,410 lb) for bolts tightened at 229 N-m (170 lb-ft)).

- The long-term loss of tension, which normally stabilizes after 30 days for bolts without wire mesh or a plywood washer, continues to increase slightly after this time for bolts with a plywood washer and wire mesh. On the other hand, the amplitude of these tension losses remains about the same in both cases. The tests described in Appendix 6 show a loss of tension from 3.5 tonnes (7,700 lb) to 2.0 tonnes (4,400 lb), and from 4.9 tonnes (10,780 lb) to 3.6 tonnes (7,920 lb) for the two situations (with and without wire mesh and plywood washer).

The wood blocks are pieces of pine generally measuring 406 x 127 x 76 mm (16" x 5" x 3"). These are used in some mines between the head plate and the wall to extend the support surface of the bolt. When they break, they also serve as warning devices when ground movements occur. In most mines, wood blocks are used only in temporary locations such as stopes. Their use in permanent locations is not recommended because, over the long term, they may cause excessive loss of tension in the bolts. There are no test results to confirm this loss of tension; however, it probably does develop if we take into account that the blocks used are generally much more compressible than the thin plywood washers described above, and that the latter already cause a certain loss of tension that continues beyond 30 days after installation.

3.1.6 Steel Straps

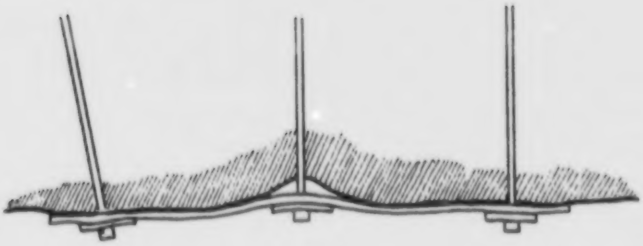


Steel straps are generally 102 mm (4") wide, and come in various lengths of 0.9, 1.2, 1.8 or 2.4 m (3, 4, 6, or 8 ft). Their thickness is generally 6.3 mm (1/4") or less.

They have perforations, three circular holes or three elongated slits, and are used for particular applications, such as holding a rock block in the roof of a gallery or reinforcing the walls of a pillar that is crumbling. Some mines use them systematically in the roof or walls of galleries to increase the support surface of the bolts. This practice is particularly common in the United States, where the use of steel mats of various lengths, 203 mm (8") wide, and 3.2 mm (1/8") thick, is widespread.

Mines located in very schistous grounds and where the schistosity is vertical, can use steel strapping in the walls to increase the support surface of the bolts and control the convergence of walls. Without strapping, in this type of ground, it is common to find that the head of the bolt gradually sinks into the rock, particularly when the bolts are used with small head plates.

The use of strapping is highly recommended in the situations described above. Their use is not recommended for grounds with many irregularities in the roof and walls of the galleries, particularly in the case of straps 6.3 mm (1/4") thick. The relative stiffness of this type of strap prevents it from adapting exactly to the shape of the irregularities, even after the bolts are driven in very tightly, and it is common to find empty spaces between the strap and the wall, as in the drawing below.



Under these circumstances the tension in the bolt can relax very quickly, which means that it will lose most of its effectiveness. The thinner straps 3.2 mm, 4.8 mm (1/8", 3/16") should be used when they can be expected to bend to accommodate the shape of the surface to which they are bolted.

3.1.7 Wire Mesh

A protective wire mesh is often installed in the roof of the galleries and kept in place by rock bolts.

There are two types of wire meshes: a welded wire mesh, similar to that used as reinforcement for concrete slabs, and a chain-link mesh (30). They are available in various mesh sizes, 51 x 51 mm (2" x 2"), 51 x 102 mm (2" x 4"), 102 x 102 mm (4" x 4") and gauge sizes no. 12: 2.70 mm (0.106"); no. 9: 3.76 mm (0.148"); no. 6: 4.88 mm (0.192"); no. 4: 5.72 mm (0.225").

Protective wire meshes have a relatively high load-bearing capacity. The loading tests discussed in Appendix 8 make it possible to draw the following conclusions:

- The load-bearing capacity of the main wire meshes used in mines is sufficient to carry the weight of the ground that is likely to detach within a rectangle measuring 1.22 x 1.22 m (4 x 4 ft), which is the size of the installation grid generally used for the bolts that serve to hold it. For a welded wire mesh, the measured load-bearing capacities vary from 13.8 kN (3,100 lb) (no. 12 gauge wire mesh measuring 102 x 51 mm (4" x 2")) to 36.0 kN (8,100 lb) (no. 4 gauge mesh measuring 102 x 102 mm (4" x 4")). For a no. 9 gauge chain-link mesh measuring 51 x 51 mm (2" x 2"), the load-bearing capacity is 28.0 kN (6,300 lb).

- For all welded wire meshes, the maximum load-bearing capacity is obtained when displacement is about 280 mm (11"). This displacement can be 406 mm (16") for chain-link meshes. During the underground inspection of a wire mesh, this much displacement may serve as a warning that the mesh is too heavily loaded, and that it has become necessary to replace it.

- The maximum load-bearing capacity of a wire mesh depends upon the quality of the installation of the bolts and head plates used to hold it. Tests reported in Appendix 8 have shown that the first wires to fail are those that are in contact with the bolts and head plates.

We can distinguish two methods that can be used to install the wire mesh with the rock bolts (Figure 11). In the first method, shown on the left side of the figure, double-threaded (DT) bolts are installed as the gallery advances and the wire mesh is added during a second stage by screwing a nut on the threaded section that sticks out; this stage includes the installation of a second head plate and a plywood washer.

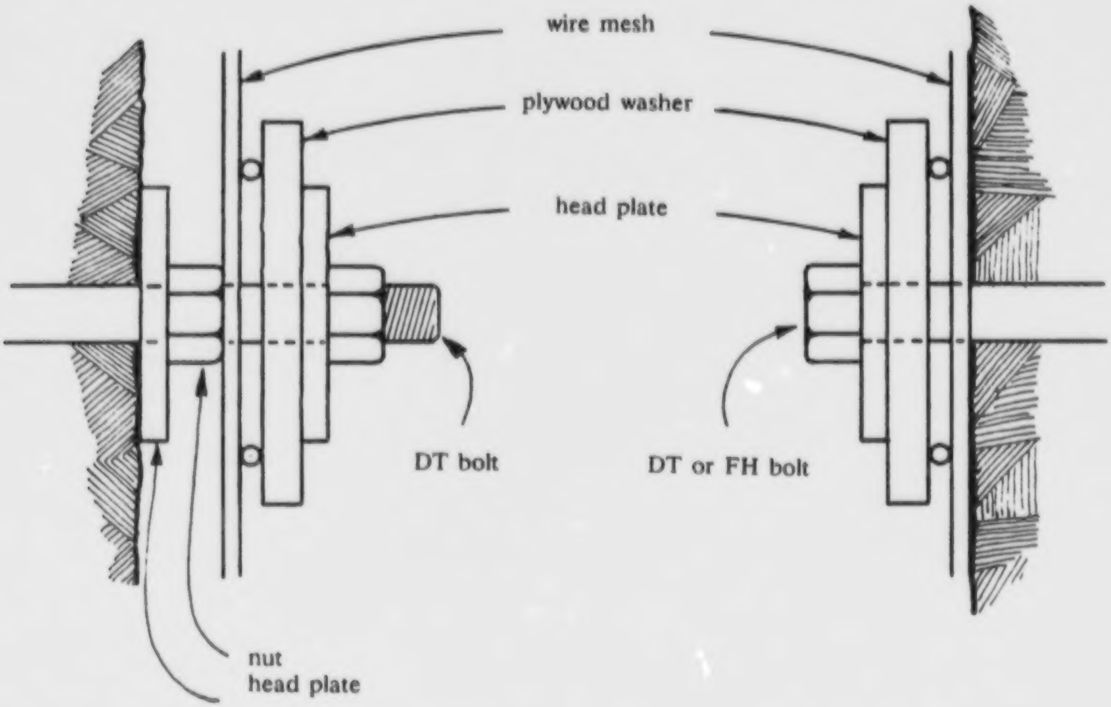


Fig. 11 - Two methods that can be used to install the wire mesh

The second method, shown on the right side of Figure 11, consists of installing the wire mesh at the same time as the bolts are installed. This method may use either double-threaded (DT) or forged head (FH) bolts. In this second method, it is important to take into account the threaded length of the bolt barrels, that is 229 mm (9") counting the two ends for the DT bolts and only 114 mm (4.5") for the FH bolts. During installation, the operator often starts to tighten the bolt before the wire mesh is flush against the wall, and uses the tightening effect to bring the mesh flush with the wall. Thus, the threaded wedge of the expansion shell of the FH bolt runs the risk of being brought quickly into contact with the non-threaded section of the bolt, and the holding capacity of the expansion shell will be consequently reduced. Moreover, if the shell is not equipped with the pop-out device described in Section 3.1.1 (Figure 12), the allowable tightening length (before the bolt comes in contact with the bail and forces the segments back unevenly see Figure 13), is even lower than the previously mentioned 114 mm (4.5") length, and is often only 51 mm (2").

For DT bolts, the allowable tightening length is longer. The tests discussed in Appendix 9 show that, during tightening, the exposed nut at one end and the threaded wedge of the expansion shell at the other end tend to turn and advance simultaneously the same distance. The allowable tightening length before the non-threaded section of the bolt comes in contact with the anchor bail is then increased to 102 mm (4"), unless the bail is equipped with a pop-out device, in which case the allowable tightening length would be even higher.

In summary, it is necessary to take the following precautions when installing a wire mesh using the second method:

- With DT bolts, do not start to tighten until the mesh has been pushed to less than 102 mm (4") from the wall.
- With FH bolts, do not start to tighten until the mesh has been pushed to less than 51 mm (2") from the wall.
- It is preferable to use a bail expansion shell equipped with a pop-out safety device.

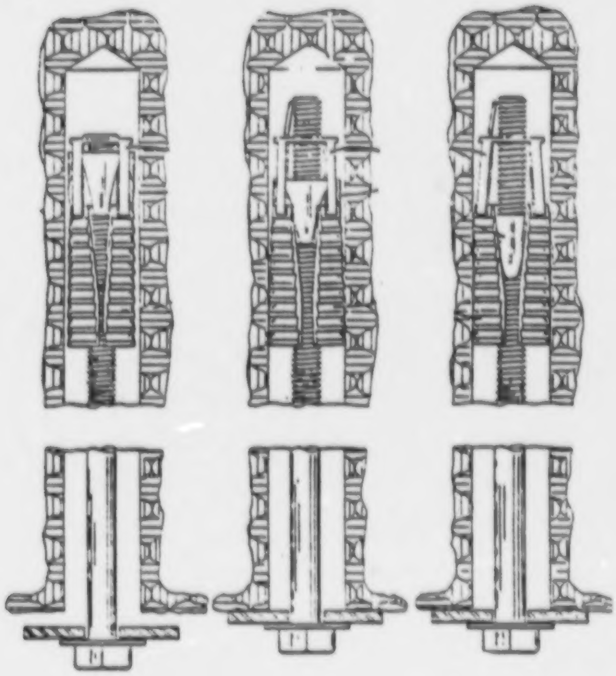


Fig. 12 - Operating principle of the pop-out device used on the bail of the expansion shell

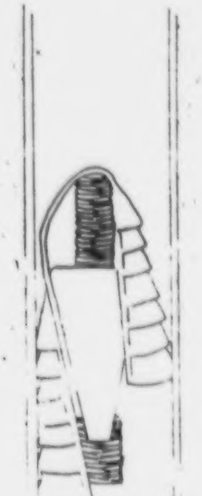


Fig. 13 - The segments are unevenly pushed back when the bolt is tightened before the mesh is resting against the wall

3.1.8 Loss of Tension in Mechanically Anchored Bolts

After the initial tightening, the level of tension in the bolts may vary, both increasing and decreasing. There are many causes of these variations, and some of them have already been discussed in sections 3.1.3 and 3.1.5 (use of head plates that are too thick or domed, use of plywood washers). An overview of these causes is shown in the table below.

Causes of a loss of tension	Causes of an increase in tension
• The initial installed load torque is too low	• convergence of the ground
• slip of expansion shell	• corrosion of the expansion shell and plate (the installed load torque increases because of the higher friction, but tension does not increase)
• bending of the plate when it is improperly resting against the wall	
• wood blocks or washers	
• blasting vibrations	
• crumbling of the rock under the head plate	

These causes may act in opposition to each other, and it is difficult to predict in which direction the tension will finally vary. For this reason, we recommend periodic verifications or, if this is not possible, the retightening of bolts in which tension has dropped. The verification and retightening can be carried out with a torque wrench, but keep in mind that the installed load torque is only roughly proportional to the tension in the bolt (wide scatter in the results of the tests discussed in Appendix 6; lower tension when the bolts are installed at an angle in relation to the wall; corrosion, which increases friction in the expansion shell). An example of a verification and bolt retightening campaign carried out in a mine to determine the effect of blasting vibrations on loss of tension in mechanically anchored bolts is shown in Appendix 10. It appears that the loss of tension in the bolts due to blasting is low since the initial installed load torque of 248 N–m (184 lb–ft) is only reduced to an installed load torque of 236 N–m (175 lb–ft) after blasting. However, the loss may have been much higher if the initial installed load torque was lower, for example 135 to 162 N–m (100 to 120 lb–ft).

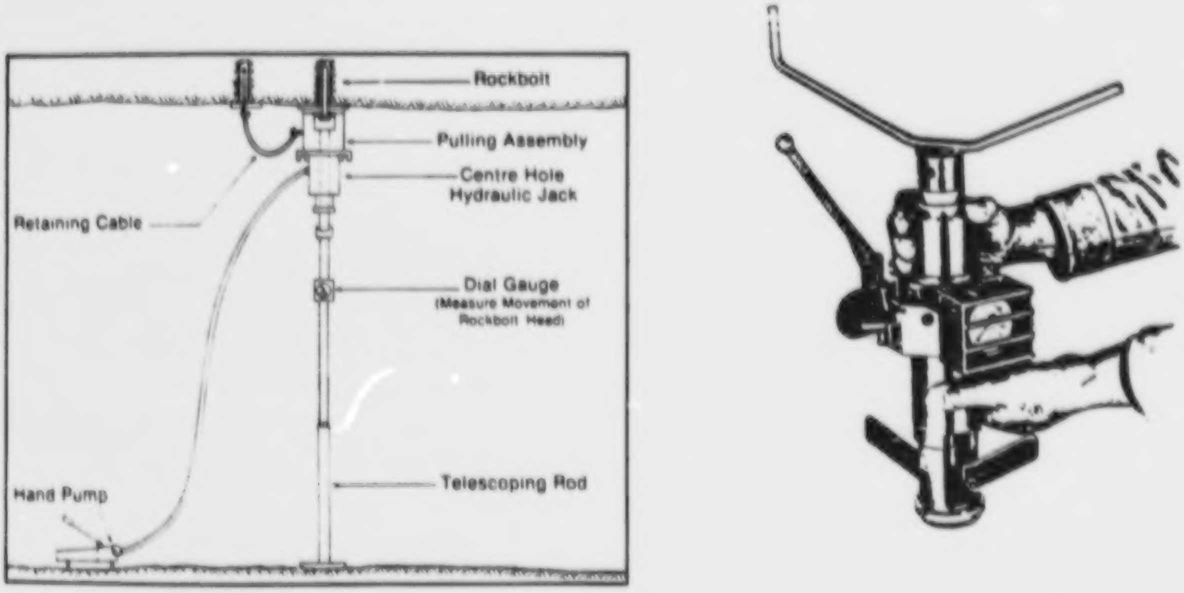
3.1.9 Results of Pull-out Tests to Verify the Anchoring Quality of Bolts (including the effect of the drill hole diameter)

Bolt pull-out tests are carried out with one of the two loading devices shown in Figure 14 (31). The first makes it possible to record the complete applied load curve as a function of elongation to rupture; the second is used mainly to verify the initial tension in the bolt and to increase the load to a predetermined level. Moreover, we generally measure the diameter of the drill hole in the location where the expansion shell will be installed using the gauge shown in Figure 15 (31).

The procedures used to carry out these tests are described in the literature (32, 33). It is possible to test any DT bolt in a gallery, but FH bolts must be equipped with an adaptor at the time of installation to make it possible to carry out subsequent tests. There are no methods to test FH bolts chosen at random in a gallery.

Appendix 11 shows the results of pull-out tests in mechanically anchored bolts carried out by three different investigators in Ontario. The main conclusions derived from these tests are the following:

- The main parameter that controls the holding quality of the expansion shell is the diameter of the drill hole. Manufacturers' recommendations regarding allowable tolerances, +1.6 mm (+1/16") to -0.8 mm (-1/32") must be complied with, and the use of a diameter gauge to verify them periodically should be encouraged. For example, the following anchoring losses were demonstrated for expansion shells used with drill holes with a nominal diameter of 31.75 mm (1 1/4").


Rockbolt
Pulling Assembly
Centre Hole
Hydraulic Jack
Retaining Cable
Dial Gauge
(Measure Movement of
Rockbolt Head)
Telescoping Rod
Hand Pump

a) Bolt pull-out device with elongation measure

b) Roof bolt torque tensioner

Fig. 14 - Devices used to carry out pull-out tests in mechanically anchored bolts. From reference (31).

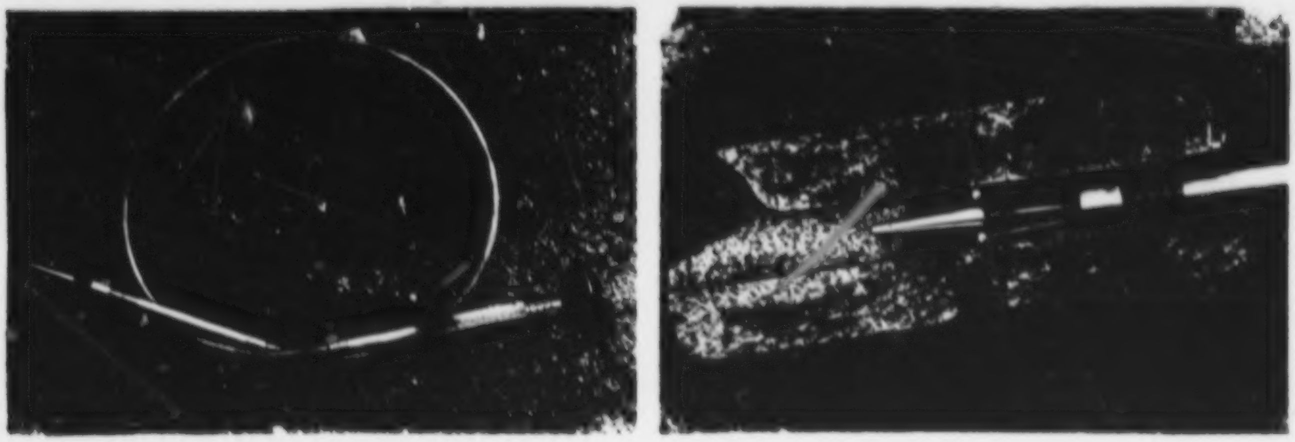


Fig. 15 - Gauge used to measure the diameter of a drill hole. From reference (31).

Test	Diameter of the drill hole (in)	Load exceeding the yield strength of steel (lb)	Pull-out force (lb)	Difference in relation to the nominal diameter of 1 1/4"
Inco	1 1/4	-	28,000	0
	1 3/8	-	9,000 to 16,000	+3/16
Ministry of Labour (Ontario)	1 7/32 to 1 9/32	18,000	-	-1/32 to +1/32
	1 3/8	2,800 to 7,000*	-	+3/16

*The tests were stopped before exceeding the yield strength of steel.

- The results of the tests below demonstrate that the performance of the FS3BL expansion shells (for holes of 31.25 to 34.9 mm (1 1/4" to 1 3/8")) is satisfactory within the limits of these diameters.

Test	Diameter of the drill hole (in)	Load exceeding the yield strength of steel (lb)	Pull-out force (lb)
Inco	1 1/4	-	28,000
	1 3/8	-	9,000 to 16,000
Stelco	1 1/4	17,000	-
	1 3/8	16,000 to 17,000	-

Little is known about the tolerance of the diameter of the drill hole above the recommended 34.9 mm (1 3/8") for use with these universal shells. On the other hand, the tolerance below the diameter of 31.75 mm (1 1/4") is certainly -0.8 mm (-1/32"), as in the case of shells used for 31.75 mm (1 1/4") holes, since the diameters of all these shells before insertion in the drill holes are about the same (30.2 mm, 1 3/16").

- Although manufacturers guarantee bolts with a diameter of 15.9 mm (5/8") a minimum rupture-load of 100.4 kN (22,600 lb) and a load exceeding the yield strength of steel of 60.3 kN (13,560 lb), most of the tests produced higher values, about 124.4 kN (28,000 lb) for the rupture load and 80.0 kN (18,000 lb) for the load exceeding the yield strength of steel.

- Some test results show that the pull-out load may drop to 97.8 kN (22,000 lb) when the bolts are anchored in a massive sulphide rock mass. Special attention must be paid when verifying the anchoring in this type of ground.
- The Ministry of Labour of Ontario has carried out about 250 bolt pull-out tests since 1978, in parallel with the introduction of provisions to Ontario regulations on rock bolting. The criterion adopted by the Ministry to determine whether a test has been satisfactory is that the expansion shell should not slip by more than 1.6 mm per tonne (0.028" per 1,000 lb) of applied tension.

Most of the bolts tested met this slip criterion or were only slightly off when they were installed in a drill hole whose diameter complied with the tolerances recommended by the manufacturers. It seems that the type of ground and the type of expansion shell have only a very small effect on the slip value.

Apart from making it possible to measure this slip value, the pull-out tests can be used to confirm whether the anchoring is good enough to support the load exceeding the yield strength of the steel used to make the bolt, that is, 80 kN (18,000 lb). As a general rule, the tests are not continued beyond this value, because of the risks associated with the sudden rupture of the bolt.

- The test results obtained by the Ministry of Labour of Ontario help emphasize a little known point on the performance of bolts equipped with expansion shells: the value of the installed load torque has no effect on the results of the pull-out tests. The interpretation of 113 tests carried out in more than 30 mines and described in Appendix 11 is summarized in the table below:

Number of bolts	Average installed load torque (lb-ft)	Average diameter of the drill holes (in)	Initial tension (tonnes)	Load exceeding the yield strength of steel (tonnes)	Anchor slip (in/1,000 lb)
46	93.8	1.30	3.0	9.4	0.0292
40	170.4	1.29	3.6	9.0	0.0295
27	211.9	1.29	3.8	9.2	0.0273

We can observe on the table that, for all practical purposes, the limit values of the tests and the anchor slips as well as the standard deviations determined in the appendix are identical for the three values of the installed load torque.

Since the installed load torque recommended by the manufacturers is 230 N-m (170 lb-ft), this value may continue to be used as a point of reference; however, bolts tightened using a much lower installed load torque (down to 101 N-m (75 lb-ft)) were still able to show satisfactory performance during the pull-out tests.

However, we should emphasize that the tests carried out were static tests, that is, using a relatively slow loading speed. The question still remains as to whether the same bolts with a low installed load torque would perform differently than bolts with a high installed load torque during a dynamic test that would simulate the falling of a rock block.

3.2 BOLTS GROUTED USING RESIN AND CEMENT

Bolts of this type are generally manufactured from reinforcement bars. They have been designed to be grouted with polyester resin cartridges previously introduced in the drill hole, or with a pumped cement grout. Even though there are other products that can be used as grout (gypsum, cement cartridges developed by the USBM (34)), these two grouting methods are the most commonly used at this time. The choice of one over the other can be made by taking into account the advantages and disadvantages of each. These are listed in the table shown in Figure 16.

3.2.1 Installation Methods

3.2.1.1 Resin-grouted Bolts

The installation method used with resin-grouted bolts varies very little from one location to another. It consists of introducing a certain number of cartridges in the drill hole and then installing the bolt using a jack leg drill or power wrench, while letting the bolt revolve to break the cartridges and ensure that the resin mixes with its catalyzer (Figure 17). The duration of this mixing stage should be neither too short nor too long because this could lower the properties of the resin, as discussed later in the text.

Polyester Resin		Cement Grout	
Advantages	Disadvantages	Advantages	Disadvantages
- bolts can be installed very quickly (cartridges) - quick setting (1 to 30 minutes) - possibility of tensioning bolts using two resins with different setting times - ease of installation. even in holes drilled upwards - very high holding power - good protection against bolt corrosion	- relatively high resin cost - average storage time (12 months) - resin vapours are toxic to the skin and eyes - the resins are flammable - decrease of mechanical properties with an increase in temperature - setting time varies with temperature	- low cost of cement - high holding power - good protection against bolt corrosion	- installation of the bolts takes longer: preparing and pumping the grout - slow setting time (one to several days) - installation more difficult in holes drilled upwards (because of the fluidity of the grout) - lack of control over the quality of the grout (segregation). and of the anchor (when the end portion of the hole is not full)

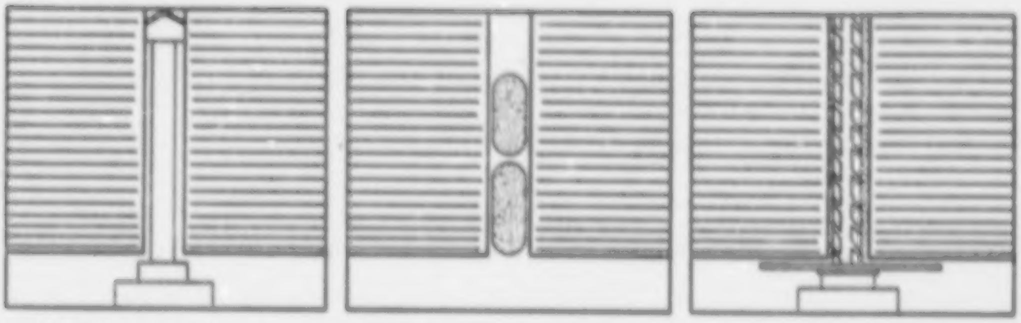
Fig. 16 - Advantages and disadvantages of bolts grouted with resin and cement grout

The diameter of the drill hole is also important and should be 6.35 mm (1/4") larger than the diameter of the bolt. This diameter minimizes the amount of resin that must be used, while it ensures a good mixture and maximum holding power in comparison with bolts installed in drill holes with smaller or larger diameters (Section 3.2.4.1). The diameter of the cartridge must also be chosen as a function of the diameter of the bolt barrel and in such a way that it makes it possible to fill the 3.2 mm (1/8") space around the bolt barrel. For example, for a 19.05 mm (3/4") bolt in a 25.4 mm (1") hole, the required cartridge diameter is 8.5 mm (1/3") (calculated as a function of the difference in cross-section between the bolt and the hole). The diameter of commercially available cartridges is generally higher than this, making it possible to install cartridges over a lower distance than the total length of the drill hole. The manufacturers provide tables that can be used to evaluate the grouted length as a function of the diameter of the bolt, the cartridge, and the hole, for a cartridge 30 cm (12") long (Figure 18).

The custom in Quebec mines tends toward the use of drill holes measuring 31.75 mm (1 1/4") with cartridges with a diameter of 28.6 mm (1 1/8"). However, it would be advantageous to use the smaller dimension cited above for the cartridge diameter, both to reduce the amount of resin required for each hole, and to ensure maximum holding power for the grouted bar.

It is possible to tension resin-grouted bolts using a quick-setting (30 seconds) cartridge in the bottom of the hole and making up the difference with slower-setting (5 minutes) cartridges. A bolt threaded at one end as well as a head plate must be used. Rotation is then used to mix the resin, and the bolt is tightened using a specially designed nut. The adaptor or nut required for tensioning is shown in Figure 19.

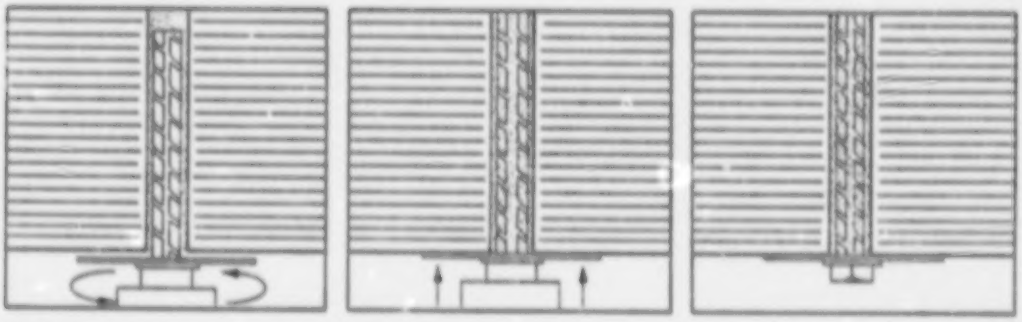
A second method to tension resin-grouted bolts is to use the combination bolts that are very widely used in the United States for coal and some metallic mines. This bolt, shown in Figure 20, consists of three parts: a ribbed bolt barrel, a smooth bolt barrel with a forged head at one end and threaded at the other end, and a coupling. The coupling contains a thin pin that is initially sufficiently strong to mix the resin when the bolt is rotated and then shears off once the resin hardens. This makes it possible to tension the bolt by tightening. Generally, one or two cartridges are used, depending on the anchoring capacity in the rock



STAGE 1: Drill a hole with a diameter of 25.4 mm (1")to the desired depth.

STAGE 2: Insert the resin cartridges in the hole. Install a plug to keep the cartridges in the hole.

STAGE 3: Push the bolt (diam. = 19.0 mm (3/4")) through the cartridges until it reaches the bottom of the hole. Do not press the plate against the wall.



STAGE 4: Spin the bolt for the time recommended by the manufacturer (10 to 20 seconds).

STAGE 5: Apply all the pressure of the device on the head of the bolt. Maintain for 20 to 30 seconds and then allow to relax.

STAGE 6: Bolt is grouted over its full length.

Fig. 17 - Procedure used to install resin-grouted bolts. From Karabin *et al.* (29)

Inches

Diameter of the cartridge

Diameter of the bolt		7/8	1	1 1/8		1 1/8		1 3/8				1 9/16		
No. 6	3/4	16"	16"	16"										
No. 7	7/8		18"	17"	16"									
No. 8	1			18"	18"	12"		16"	13"					
No. 9	1 1/8					15"	12"	20"	16"					
No. 10	1 1/4						16"		19"	16"				
No. 11	1 3/8									20"	15"	12"		
No. 14	1 5/8												12"	
		1	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8	1 1/2	1 5/8	1 3/4	1 7/8	2	2 1/4	

Diameter of the drill hole

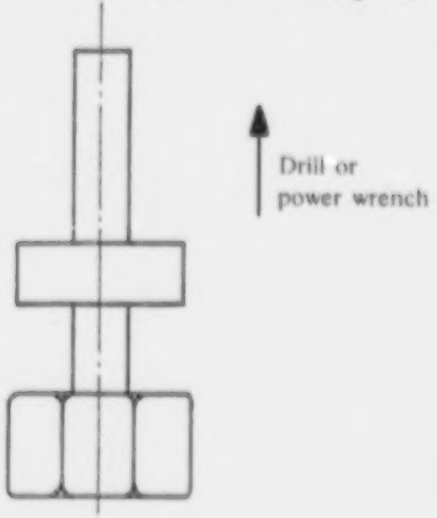
Millimeters

Diameter of the cartridge

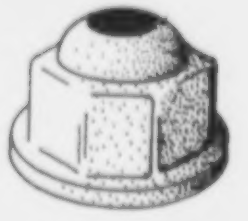
Diameter of the bolt	22	25	28		32		36				40		
19 mm	310												
20 mm	406	406	406	330									
25 mm			508	457	305		406	305					
30 mm					432	356	533	457	432				
35 mm									508	381	305		
45 mm												330	
	25	30	32	35	38	40	38	40	45	48	51	57	

Diameter of the drill hole

Fig. 18 - Table used to evaluate the grouted length with a resin cartridge 30 cm (12") long (35).



Adaptor used to rotate the bolt (Tightening with another nut already screwed in on bolt)



Domed-head nut used to rotate and tighten the bolt

Fig. 19 - Adaptor or nut used to install and tension resin-grouted bolts

(see Section 3.2.4.2). Appendix 12 describes the results of tests carried out in the United States by the Mines Safety and Health Administration (MSHA) to compare the short- and medium-term performances (loss of anchoring capacity) of combination and mechanically anchored bolts.

3.2.1.2 Cement-grouted Bolts

There are various installation methods, and these are discussed below.

- Very long holes drilled upward: operators should use a grout tube passing through the head plate and attached to the bolt barrel over a short length (Figure 21), and use an air return tube attached over the entire length of the bolt barrel and stopping under the plate, or going through it if air return will be controlled by putting the tube in a bucket of water. The tubes must be made of semi-stiff plastic, with walls at least 3.2 mm (1/8") thick, particularly in the case of the air tube, to ensure that it will not be closed by the pressure of the grout in the hole. A putty plug must be used at the borehole collar to prevent the grout from falling out during pumping.
- Short and long holes drilled downward: It is possible to use a single grout tube attached to the bolt over its entire length to pump the grout from the bottom to the top.
- Short- and medium-length holes drilled upward: It is not necessary to use an air return tube. A simple technique consists of pumping the cement grout directly into the hole by inserting the grout tube down to the bottom and then withdrawing it as the hole is filled. When the hole is full, the bolt bar is pushed inside (Figure 22), and then the putty plug is put in.

Although the water/cement ratio is chosen to keep the grout to be pumped at an acceptable consistency, the ratio should be kept in the vicinity of 0.4 (Section 3.2.5.1) to make it possible to obtain the maximum compressive strength value after curing. To keep this low water/cement ratio and still obtain the required consistency, a fluidization additive should be used, as well as an expansion additive (aluminum powder) to counteract the natural shrinkage of the cement. The grout can also be mixed with mineral additives such as crushed slag with a high silica content, blast furnace silica ashes, clinker dust from cement plants, or flue dust from power stations. Each of these additives can increase the compressive strength of the grout, without changing the initial water/cement ratio.

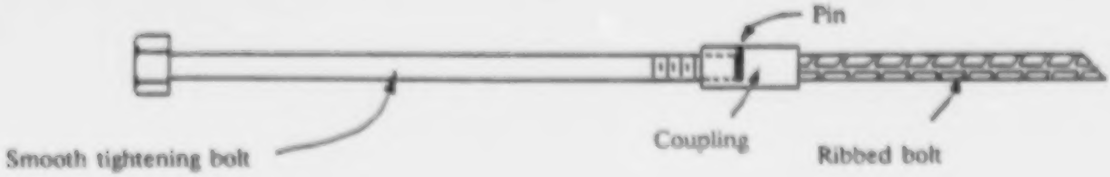

Pin
Smooth tightening bolt
Coupling
Ribbed bolt

Fig. 20 – Combination bolt

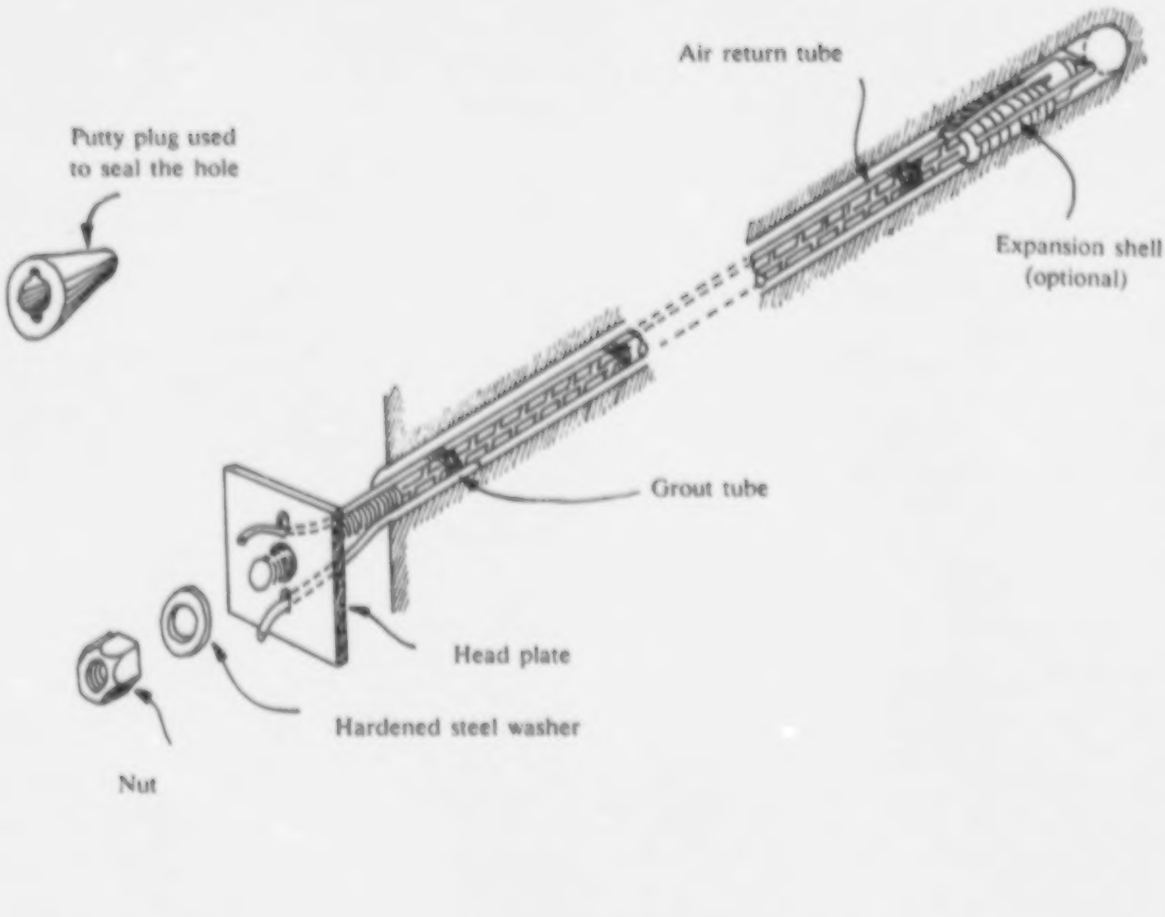

Air return tube
Putty plug used
to seal the hole
Expansion shell
(optional)
Grout tube
Head plate
Hardened steel washer
Nut

Fig. 21 – Injection and air return tubes attached along the bolt

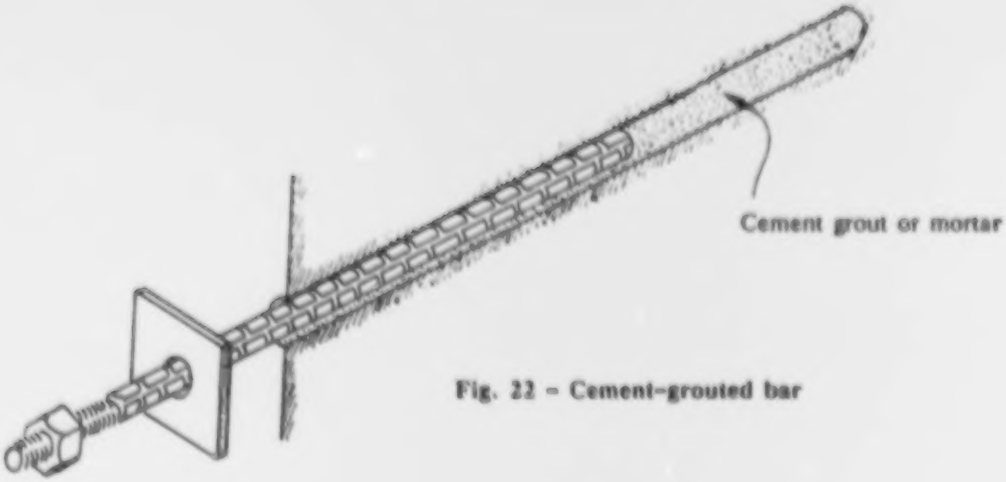
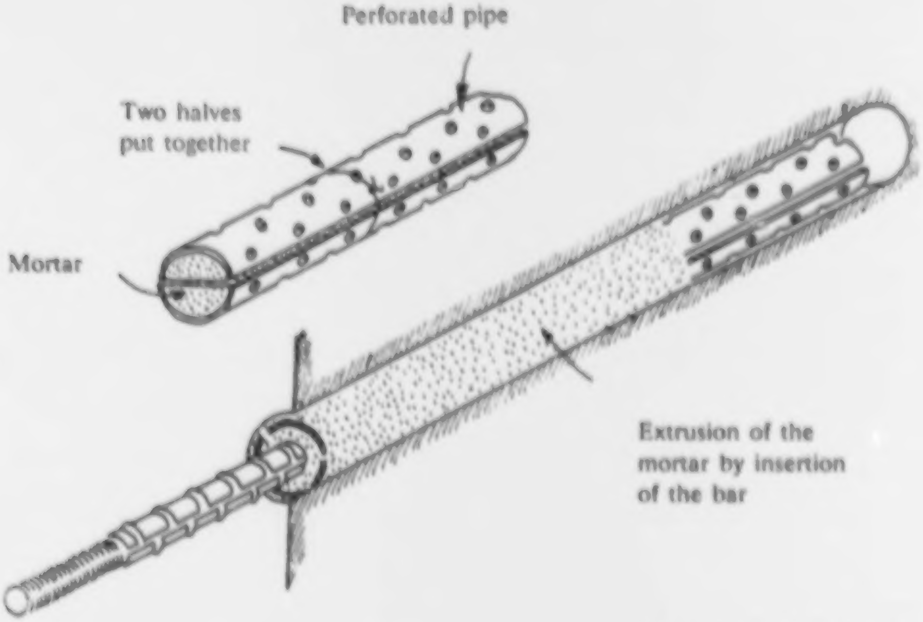

Cement grout or mortar

Fig. 22 - Cement-grouted bar


Perforated pipe
Two halves put together
Mortar
Extrusion of the mortar by insertion of the bar

Recommended diameters:

Bar		Drill hole		Pipe	
3/4 in	19 mm	1 1/4 in	32 mm	1 1/16 in	27 mm
1	25	1 1/2	38	1 1/4	32
1 1/8	29	1 3/4	44	1 1/2	38
1 1/4	32	2	51	1 3/4	44
1 3/8	35	2 1/4	57	2	51

Fig. 23 - Perfobolt (patented) system used to install a cement-grouted bolt

The choice of the diameter of the drill hole is not as important as for resin-grouted bolts. The hole diameter can be between 12.7 mm (1/2") and 25.4 mm (1") larger than the diameter of the bar.

Nevertheless, the installation method based on the direct pumping of the grout does not ensure the good quality of the anchor at the bottom of the hole, mainly because of the unavoidable subsiding of the grout in the hole, which tends to take it back toward the opening. A way to get around this difficulty, and one that is particularly appropriate for supporting permanent locations, is the Perfobolt system (7, 21) shown in Figure 23. In this method, a perforated pipe cut into two halves is used to contain the cement mortar; then the two halves are reclosed, attached together using a pin, and inserted into the drill hole. The bolt is then pushed inside the pipe; this forces the mortar out and ensures that the entire hole is grouted. Because mortar subsides less than cement grout, it is possible to ensure a good seal over the entire length of the bolt. This method is currently used in civil engineering installations in Scandinavia, where it is protected by a patent.

The recommended diameters of the hole, the bar, and the pipe are shown in Figure 23 (21). As in the previous case, the difference between the diameters of the bar and those of the hole is 12.7 mm (1/2") to 25.4 mm (1"). The composition (in weight) of a mortar with adequate subsidence is as follows:

Cement:	1 part
Sand:	1 part
Water:	0.3 parts
Fluidization additive:	0.007 parts
Expansion additive:	0.00015 to 0.0007 (up to 15% expansion)

3.2.2 Properties of the Bolt Barrels

As already discussed, the bolt barrels generally consist of ribbed reinforcing bars. These bars are available in various diameters; the most commonly used are no. 6: 19.0 mm (3/4") and no. 7: 22.2 mm (7/8") bars, whose properties are listed below (28).

Yield strength: 58,000 psi (400 MPa)

Ultimate strength: 80,000 psi (550 MPa)

Load exceeding the yield strength (diam. 3/4"): 20,040 lb (89.1 kN)

Load exceeding the yield strength (diam. 7/8"): 27,276 lb (121.2 kN)

Load at rupture (diam. 3/4"): 26,750 lb (118.9 kN)

Load at rupture (diam. 7/8"): 36,369 lb (161.6 kN)

Percentage of elongation at rupture: not available

The loads exceeding the yield strength and the loads at rupture shown on the table take into account the decrease in the effective cross-section of the bars in the traction area of the threads.

It is also possible to use reinforcing bars in metric sizes, particularly the bar no. 20M with a diameter of 19.5 mm, which is only 3 per cent larger than that of bar no. 6, and is made of the same grade of steel.

The forged heads or threads are added to the bolt barrels in the workshop. At the same time, it is possible to round off the end of the threaded section to make it easier to install the wire-mesh retention nut and to taper the other end to facilitate the insertion of the bolt barrel in the drill hole.

In some applications, it may be necessary to use smooth rather than ribbed bolt barrels. Smooth barrels reduce the stiffness of resin-grouted bolts installed in grounds where wall convergence is high (schistous grounds at medium depths, or deep underground) and where many bolt heads break because of excess tension on the bolt; smooth barrels may also be necessary in locations affected by rock bursts (Sections 3.4.3, 4.3.2, and 4.3.3).

Another type of bolt can also be used - a threaded bar (Dywidag bar) that has the advantage that it can be assembled end-to-end using a threaded coupling (Figure 24). This bar allows the insertion of long bolts in small galleries or work sites. These are available in various diameters and grades of steel, some of which have very high strength characteristics (150,000 psi; 1,030 MPa) that make them suitable for applications where high holding power and tensioning (by tightening the nut) are necessary.

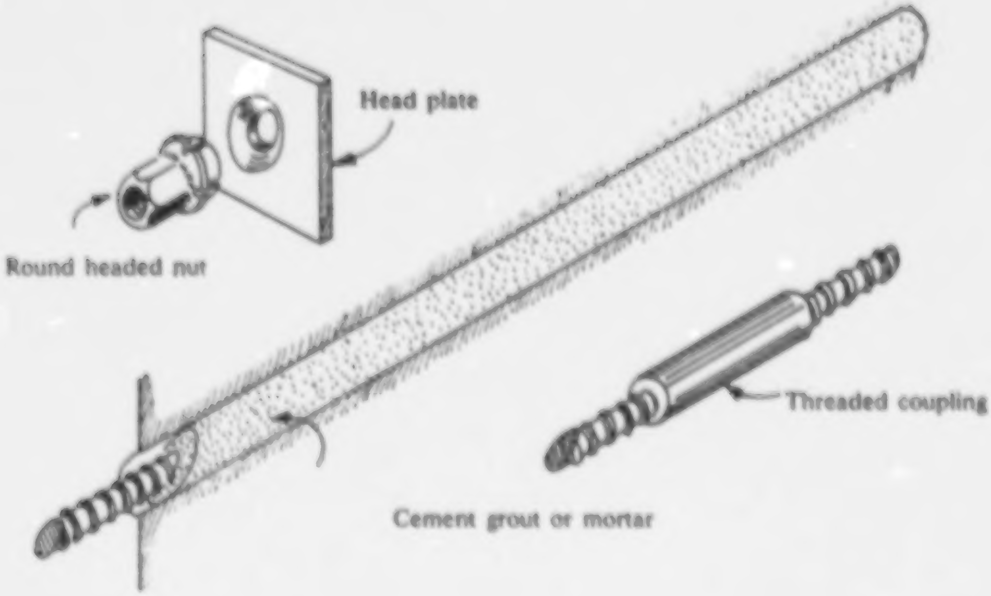

Head plate
Round headed nut
Threaded coupling
Cement grout or mortar

Fig. 24 - Threaded Dywidag bar with tightening screw and coupling

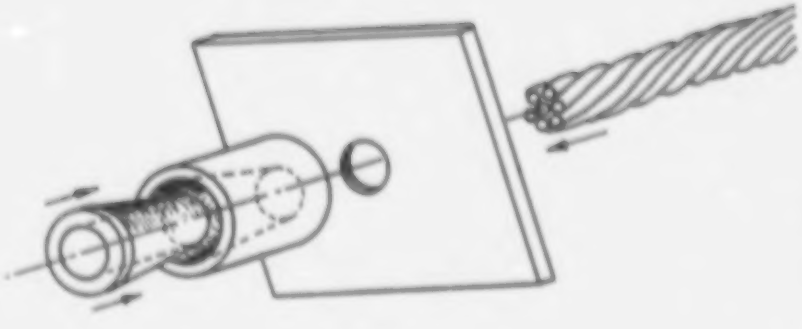


Fig. 25 - Fastening device used to attach a head plate to a cable bolt

Some mines have experimented successfully with high-strength stranded cablebolts with a diameter of 15.9 mm (5/8") (load at rupture: 257.7 kN (58,000 lb)) as gallery support devices. These cables can be cut to the desired length and then grouted with cement using one of the previously described methods. It is also possible to add a head plate that is held by a specially designed fastener (Figure 25).

Finally, we should mention the 1978 development by the U.S. Bureau of Mines (34) of a bolt drilling and "pumping" device mounted on a commercial bolting jumbo. Even though initially developed for bolting the roof of thin coal seams where the length of the bolts exceeded the height of the galleries, this device could be useful in mines trying to deal with a problem of severe corrosion of bolts because of the presence of very acidic underground water. The holes are drilled with a flexible bit, and then a fibreglass reinforcement is inserted and injected with an epoxy resin. This device was tested in 1982 in a mine in Pennsylvania.

3.2.3 Chemical and Mechanical Characteristics of the Resins

The resins used for mining applications (12) are thermosetting resins. Unlike thermoplastic resins, which build polymeric chains of relatively low strength, thermosetting resins can obtain high strength values.

A thermosetting resin is formed by an exothermic chemical reaction between the resin and a catalyzer that acts as the hardening agent. Even though epoxy resins have already been tested, polyester resins are the most widely used at present, because they are less expensive and make it possible to obtain faster hardening at mining temperatures.

An inherent disadvantage of polyester resins is that hardening is accompanied by shrinkage. Pure resin may shrink by 8% to 17%, so that a filler consisting of quartz or calcite is generally used to reduce shrinkage to less than 1%. This filler also makes it possible to reduce the unit cost of the cartridges; however, it also reduces the natural ductility of the resin and makes it more fragile.

Manufacturers (Celtite, Dupont, Ground Control) produce resins with setting times ranging from 1 to 30 minutes. However, the setting time varies with temperature (see Figure 26, for a Dupont [Fastloc] resin with a nominal setting time of 2 minutes at 15°C).

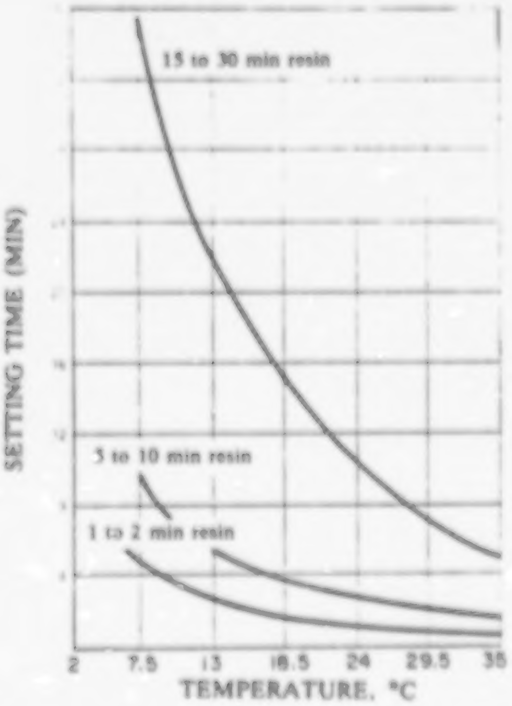


Fig. 26 – Variation in setting time as a function of temperature for polyester resin (36)

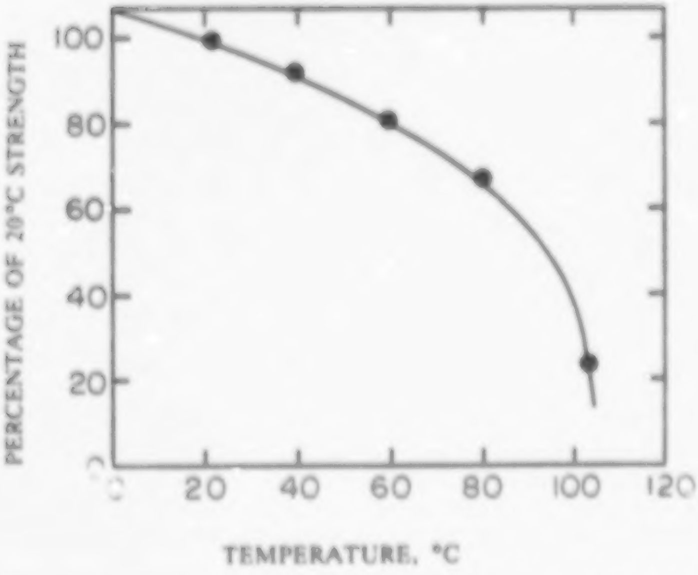


Fig. 27 – Variation in the compressive strength of the polyester resin as a function of temperature (37)

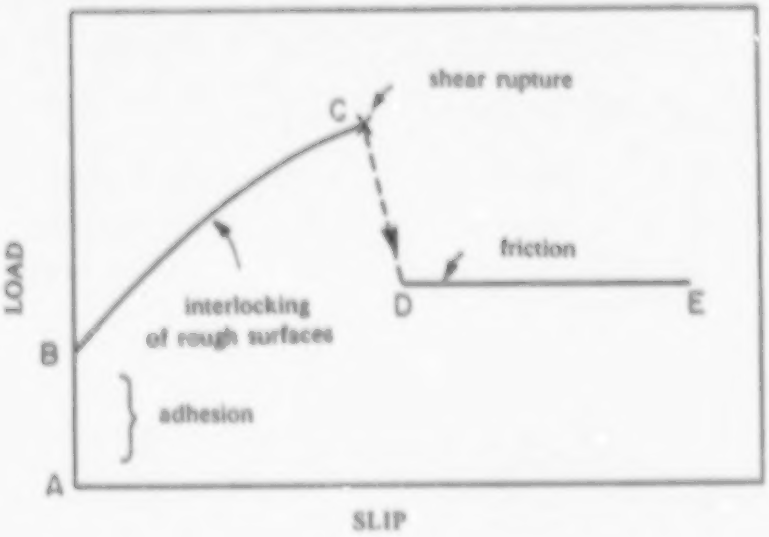
Depending on the manufacturer, there is a wide variation in the physical and mechanical properties of resins. Typical values are shown in the table below (12).

Density	Tensile strength (MPa)	Compressive strength (MPa)	Shear strength (MPa)	Modulus of elasticity (MPa)	Poisson's ratio	Elongation at rupture (%)
1.85	17	110	50	7,000	0.3	0.2

These properties may change considerably with temperature (see Figure 27).

3.2.4 Holding Power of Resin-Grouted Bolts

Pull-out tests may be carried out with the same devices as those shown in Figure 14. The typical shape of the pull-out curves obtained is shown below.



At the beginning of the test, the AB branch corresponds to the chemical adhesion of the resin to the bolt and the walls of the drill hole. However, this resistance is rapidly exceeded for a very short slip.

In the second stage, holding power is obtained on the BC branch through the interlocking of irregularities in the interface between the resin and the enclosing rock. A rough hole and a ribbed bar are more effective than a smooth hole and a smooth bar. Appendix 13 discusses the results of tests confirming that interlocking of irregularities is the main mechanism that explains holding power.

Shear rupture, or eventually the rupture of the bolt, is finally reached at point C. This rupture can be explained by one of the five modes shown in Figure 28, where various parameters, such as the roughness of the hole and bolt, the hardness of the enclosing rock, and the diameter of the drill hole may all play a role in determining the type of rupture obtained, and consequently on the value of the holding power. Holding power is at a maximum when grouting conditions are such that the first mode (that is, the rupture of the bolt) occurs. It is important to choose the resin-grouted bolting parameters in such a way that, during pull-out tests, we can obtain the rupture of the bolt rather than one of the other modes.

Beyond shear rupture, there is a drop in strength along the CD branch, to an intermediate value that corresponds to friction along the slip interface (bolt-resin, etc.). Even though this friction is present, it is not always demonstrated during the tests, because these are often stopped when maximum strength is reached.

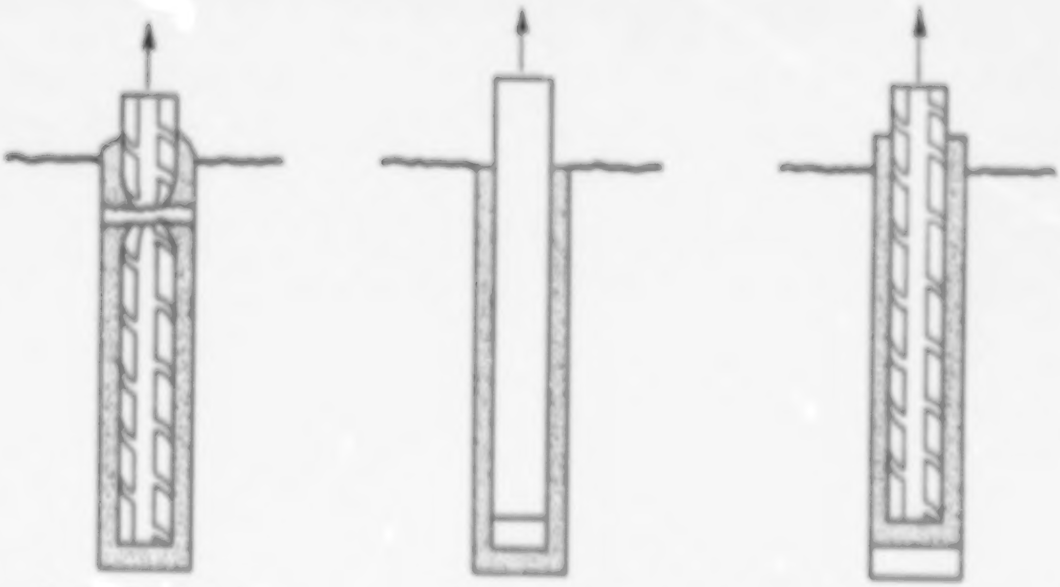
The effect of the various parameters listed above on holding power is discussed in the following sections.

3.2.4.1 Relationship Between the Diameter of the Drill Hole and the Pull-out Force

The series of tests discussed in Appendix 14 makes it possible to derive the following conclusions:

- Bars grouted in drill holes with a diameter that is more than 6.35 mm (1/4") larger than their own diameter have a pull-out load equal to the rupture load of the bolt, and their pull-out curve shows very significant stiffness.

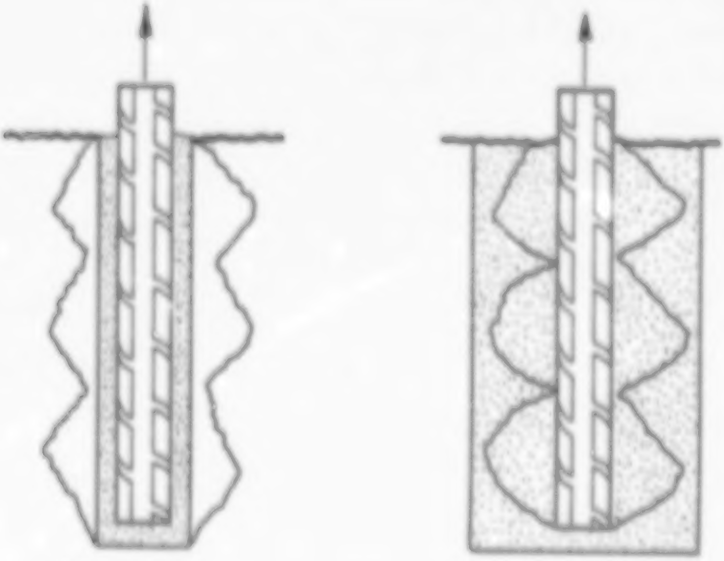
- A difference in diameter of 6.35 mm (1/4") greatly facilitates the tearing of the cartridge envelope and proper mixing of the resin.



Rupture of the bolt (ribbed bolt, rough hole walls)

Bolt–resin shear (smooth bolt)

Resin–rock shear (hard rock, smooth hole wall)



Rock shear (soft rock)

Resin shear (diameter of the hole is too large)

Fig. 28 – Various modes of shear rupture obtained during pull-out tests of a resin-grouted bolt

- Bars grouted into drill holes with a diameter that is 9.5 mm (3/8") larger than their own diameter also make it possible to reach the rupture load of the bolt, but the pull-out curve shows a stiffness that is nine times lower. This is only favourable in grounds where the wall convergence is high, because this prevents the rupture of the bolt heads.
- Bars grouted in drill holes with a diameter that is 15.9 mm (5/8") larger than their own diameter show a drop in pull-out load of 60%, with shear rupture through the resin.
- In extremely soft grounds (compressive strength of 700 to 1,500 psi (4.8 to 10.3 MPa)), a significant improvement in holding power can be obtained by increasing the diameters of both the bar and the drill hole while still maintaining a difference of 6.35 mm (1/4") between their diameters.

3.2.4.2 Relationship Between the Type of Rock and Minimum Grouting Length

It is generally accepted that the grouting distance for bolts grouted over their entire length is greatly superior to what would be required to ensure an adherence equal to the rupture load of the bolt. This is particularly true for hard rocks, where a grouting length of 30 to 60 cm (1 to 20 ft) is enough to meet this objective. The use of the combination bolt in American mines, described in Section 3.2.1.1, supports this statement.

Several pull-out tests carried out by various investigators provide a complete range of the strength values that can be obtained. These are generally expressed as adherence factors (grouting length (inches) per short ton of anchoring strength). One of the most commonly used graphs is that produced by the Celtite company (38). The graph is based on 105 pull-out tests on grouted bolts installed in five different types of rock and is valid for 25.4 mm (1") bolts grouted in 31.75 mm (1 1/4") drill holes with Selfix polyester resin. This graph is shown in Figure 29. The adherence factor is shown as a function of the compressive strength of the enclosing rock. The curve at the bottom is the one that can generally be used; however, the curve at the top represents a more conservative adherence factor and was traced to take into account that the tests carried out in sandstone (number 3, Figure 29) produced factors that were lower than those obtained with other types of rock.

By referring to this graph and assuming a load at rupture of 177.7 kN (40,000 lb) for a bolt with a diameter of 25.4 mm (1"), the grouting length required in granite with 127 MPa

(18,500 psi) would be 20 x 0.25"/short ton = 127 mm (5"). The authors then suggest an additional safety margin (152 mm (6") in hard rock, 305 mm (12") in soft rock) to take into account that the test results showed significant scatter. Thus, the grouting length required in this example is 127 mm + 152 mm = 280 mm (5" + 6" = 11").

Other pull-out tests were carried out by Gerdeen *et al.* (12). These authors found that their test results were in agreement with the Celtite graph, and this allowed them to extrapolate to bars with diameters of 19.0 mm (3/4") and 31.75 mm (1 1/4"). The recommended adherence factors are shown in the table in Figure 30. It is important to point out that the recommended values already take into account a safety factor to compensate (as noted above) for the scatter observed in the test results.

Finally, the results of other pull-out tests on various bolts grouted over their entire length or over 30 cm (1 ft) in various types of rock have been reported by Bartels *et al.* (39). Of these, the results shown below concerning holding power when resin mixing time is decreased are particularly interesting.

Resin mixing time	10 sec*	5 sec	3 sec	2 sec
Pull-out load	23,900 lb	20,900 lb	19,900 lb	4,000 lb
Comments	Bolt** breaks	Bolt pulls out	Bolt pulls out	Bolt pulls out

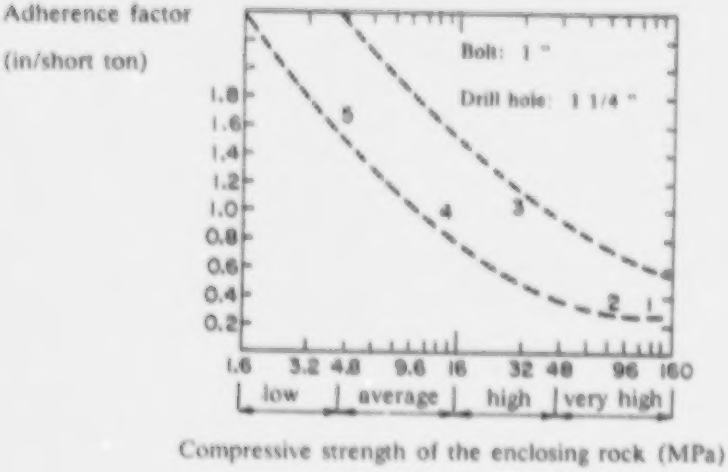
* recommended by the manufacturer

** bolt with a diameter of 19.0 mm (3/4") in a 25.4 mm (1") drill hole

3.2.5 Holding Power of Cement-grouted Bolts

There is much less data on the holding power of cement-grouted bolts than on those grouted with resin. However, some experimental results confirm that their performance during pull-out tests is similar to that described in Section 3.2.4, especially the three phases of the loading curve (adhesion, interlocking of irregularities, and friction along the slip interface) and also the five causes of rupture shown in Figure 28.

Moreover, there is a sixth cause of rupture: while the composition and quality of resin cartridges may be considered as constant during installation, the same is not true for cement grout, where the water/cement ratio may vary widely. Poor control over this parameter may



1: granite
2: limestone
3: sandstone
4: coal
5: chalk

Fig. 29 – Graph used to select the adherence factor for resin-grouted bolts with a diameter of 25.4 mm (1"). From Franklin et al. (38))

Compressive strength of the enclosing rock (psi, MPa)	Adherence factor (in/short ton)	Diameter of the bolt (in)	Diameter of the drill hole (in)
500–1,000, 3.4–6.9 (mudstone, siltstone)	3.75 3.00 2.50	0.75 1.00 1.25	1.00 1.25 1.50
1,500–3,000, 10.3–20.7 (coal, schist)	2.50 2.00 1.67	0.75 1.00 1.25	1.00 1.25 1.50
4,000–10,000, 27.6–69.0 (sandstone, limestone)	1.88 1.50 1.25	0.75 1.00 1.25	1.00 1.25 1.50

Note: Minimum grouting length: 17.8 cm (7")
For smooth holes, multiply the adherence factor by 2 to 3

Fig. 30 – Recommended adherence factors for 19.0 mm (3/4"), 25.4 mm (1"), and 31.75 (1 1/4") resin-grouted bolts. From Gerdeen *et al.* (12)

be the main cause of the loss of holding power of bolts undergoing pull-out tests. This loss of holding power is caused as much by excessive segregation in the drill hole when the mixture is too liquid, as by a hydration reaction modified by the excess water that leads to a decrease in the compressive strength of the mixture.

3.2.5.1 Relationship Between the Water/Cement Ratio and the Properties of the Cement Grout

As we mentioned in Section 3.2.1.2, the ideal water/cement ratio for a cement grout is 0.4 to 0.45. Figure 31 illustrates the effect of this ratio on the three properties of the grout that are important for grouting applications for rock bolts: compressive strength, pumping facility, and bleed capacity.

The Littlejohn *et al.* report (40) stated that the target value of the compressive strength of the grout at 28 days is 28 MPa (4060 psi). Considering Figure 31, this value could be obtained with a water/cement ratio as high as 0.53. On the other hand, bleed capacity, particularly for holes drilled upward, starts with a ratio of 0.4, and increases rapidly, leading to disastrous effects as far as corrosion protection is concerned.

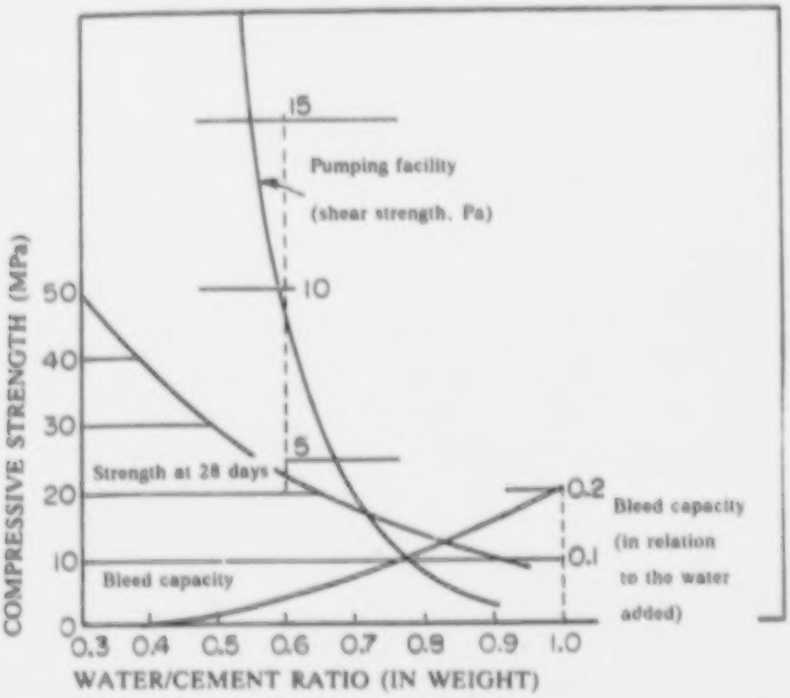
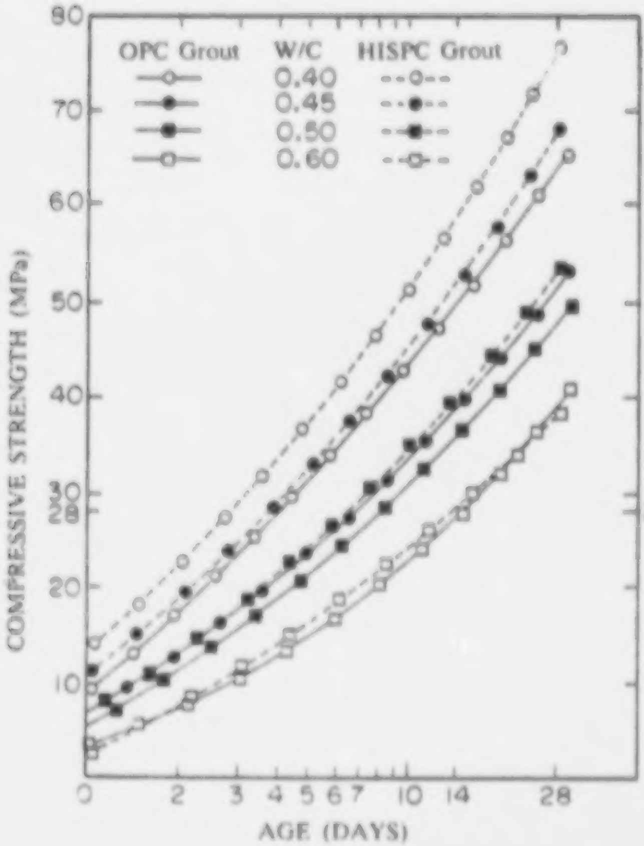


Fig. 31 - Effect of the water/cement ratio on the properties of a cement grout. From Littlejohn et al. (40)



OPC: Ordinary portland cement

HISPC: High initial strength portland cement

28 Target value of compressive strength

Fig. 32 - Effect of the water/cement ratio on the compressive strength of a cement grout as a function of its age (days). From Littlejohn et al. (40)

Moreover, for ground support applications, it is important that the cement grout reach the target compressive strength value of 28 MPa (4060 psi) as quickly as possible. Figure 32 illustrates the necessary delays with the various types of mixtures. For example, the target value is obtained in three days for a high initial-strength cement with a water/cement ratio of 0.40, and in four days for an ordinary Portland cement.

After a certain threshold, the use of a water/cement ratio that is too high may have disastrous consequences on the holding power of the bolts. For example, pull-out tests carried out in a Quebec mine (41) on cable bolts with a diameter of 15.9 mm (5/8"), grouted in 50.8 mm (2") drill holes over short lengths of 30 cm to 1.5 m (1 to 5ft), produced pull-out loads that were four times lower than those that can be expected with the help of the methods discussed in the following section. These low pull-out values are explained by a 1.0 water/cement ratio that was used at the mine.

3.2.5.2 Relationship Between the Compressive Strength of the Rock or Cement Grout and the Minimum Grouting Length

The adherence between cement grout and enclosing rock has been studied by various authors (Coates *et al.* (42), Brown (43), Ballivy *et al.* (44)), who have carried out series of pull-out tests.

According to Brown, the following grouting lengths (L) are recommended for a ribbed bar. These lengths take into account a safety factor of 2.0 to 2.5.

Sound rock: L = 30 bar diameters
Fissured rock: L = 40 bar diameters
Weathered rock: L = 60 bar diameters

On the other hand, Ballivy *et al.* carried out their pull-out test results on ribbed bars in various sedimentary and metamorphic rocks in Quebec. They then compared their results with those obtained by Brown and a few others. The results are summarized in Figure 33, from which the two following equations can be obtained:

Lower boundary: $\tau = 0.17\sqrt{f'c}$

Average: $\tau = 0.50\sqrt{f'c}$

where $f'c$: compressive strength of the cement grout or the enclosing rock (the lower of the two values), in MPa

τ : adherence strength at the grout-rock interface, in MPa

Although the use of the lower bound formula makes it possible to represent all the experimental results shown in the figure, Ballivy *et al.* (44) recommend the use of the formula which accounts for average values on Figure 33. Moreover, it is important to point out that this formula also takes into account the possibility of rupture at the bar-grout interface because, when this took place during the test, the pull-out force obtained was converted into an equivalent adherence strength that is mobilized at the grout-rock interface.

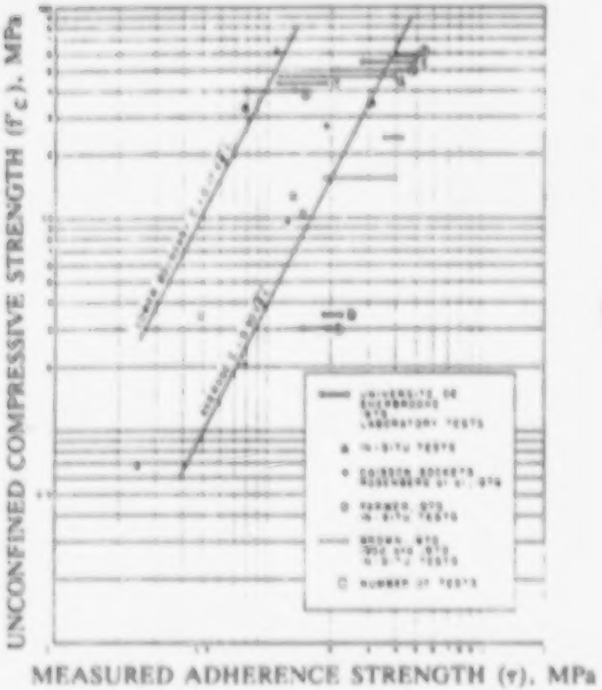


Fig. 33 – Relationship between the grout-rock adherence on the wall of the drill hole and the compression strength of the grout or rock, depending upon which is lower.
From Ballivy *et al.* (44)

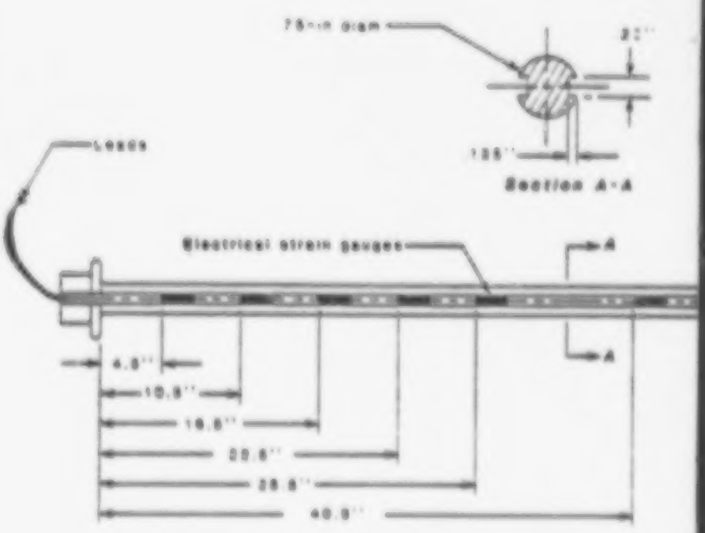

75-in diam
Leads
Section A-A
Electrical strain gauges
A
A
4.5''
10.5''
16.5''
22.5''
28.5''
40.5''

Fig. 34 – Bolt equipped with strain gauges

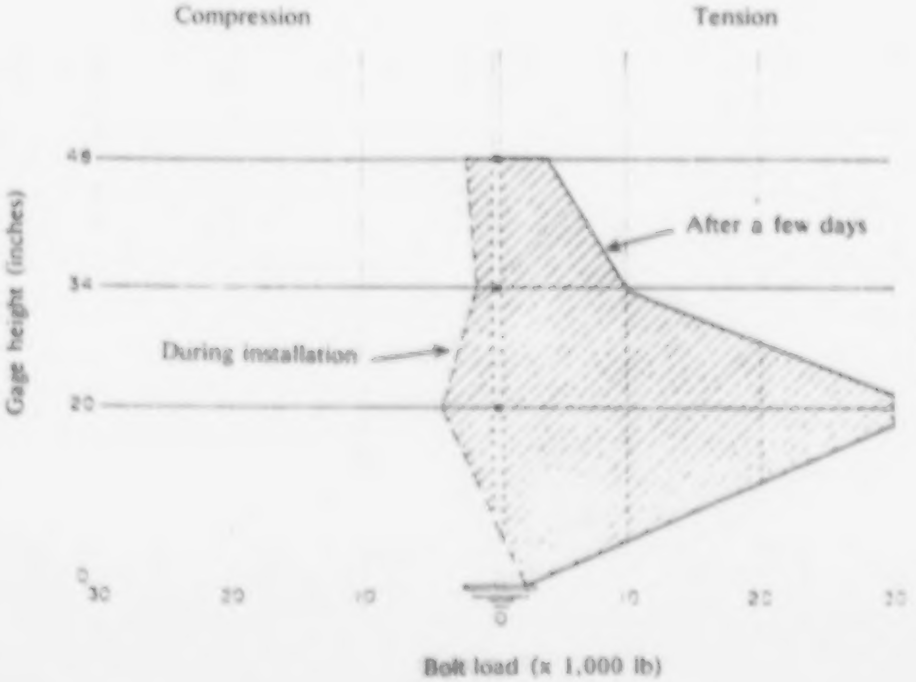


Fig. 35 – Tension profile measured along a resin-grouted bolt.
From Karabin *et al.* (29)

An example of selecting the minimum grouting length (L) of a rock bolt on the basis of the two methods is given below:

Bolt: diameter 0.019 m (3/4"), load at rupture: 118.9 kN (26,720 lb)

Cement grout compressive strength: 28 MPa

Drill hole diameter: 1 1/2" (0.0381 m)

Enclosing rock: fractured rock, compressive strength: 40 MPa.

a) Ballivy's method:

$$L(m) = \frac{\text{load at rupture (kN)}}{\tau . 10^3 . \pi . D(m)}$$

where $\tau = 0.5\sqrt{28} = 2.64$ MPa

thus $$L = \frac{118.9}{2.64 \times 10^3 \times \pi \times 0.0381} = 0.38 \text{ m}$$

b) Brown's method

$$L = 40 \times 0.019$$

$$L = 0.76 \text{ m}$$

The minimum grouting length L obtained on the basis of Ballivy's method is two times shorter than that obtained using Brown's method. This confirms that the second method takes into account a safety factor of 2.

Finally, we would like to point out that Ballivy's formula may be reversed to calculate the minimum required drill hole diameter when the grouting length is known, and when we want to guarantee that the holding power distributed over this length is at least equivalent to the load at rupture of the bolts. This situation applies particularly in the ground suspension case (Section 2.1). For example, in the situation shown in Figure 2, or, when the grouting length (L) in the limestone layer with good holding power is set at 0.4 m, the minimum drill hole diameter D can be obtained with the following formula:

$$D(m) \text{ minimum} = \frac{\text{load at rupture (kN)}}{\tau . 10^3 . \pi . L \text{ (m)}}$$

By keeping the same parameters as those listed above, we obtain:

$$D = \frac{118.9}{2.64 \times 10^3 \times \pi \times 0.4} = 0.036 \text{ m}$$

3.2.5.3 Effect of Using Mineral Additives and Fluidization and Expansion Agents in Cement Grouts

Mineral additives (silica ashes) as well as expansion (aluminum powder) and fluidization (superplastifiers) additives are often used with cement grout to improve its properties.

From the study discussed in Appendix 15, we can arrive at the following conclusions regarding the improvement in the holding power of the grouted bolts:

- The improvement in holding power obtained by adding aluminum powder is relatively low, in the vicinity of 8.5%.
- This same improvement can be increased to 24% when 10% of the cement is replaced by an equivalent weight of silica ashes.
- Better holding power is obtained with a grout that contains aluminum powder, silica ashes, and Ottawa sand, in a proportion equal to the combined weight of the cement and the silica ashes. This mixture, which has a lower prime cost than that of the cement alone, makes it possible to obtain an improvement in holding power of 50%.
- This latter mixture makes it possible to obtain a holding power that is 19% higher than what can be obtained with an epoxy resin under the test conditions described in Appendix 15.

3.2.6 Use of a Head Plate

In Quebec mines, the use of a head plate with bolts grouted with resin or cement is not common.

However, many results have shown that when head plates are added to the bolts, they become an integral part of the support and can carry a load that may become significant. For example, Cincilla *et al.* (46) used load cells to obtain measures of the load carried by head plates used with 91 untensioned resin-grouted bolts over a period of several months in two mines opened in sedimentary ground. In the first mine, the load on the plates after six months reached an average value of 84.0 kN (18,900 lb); while the head plates in the second mine carried an average load of 26.2 kN (5,900 lb).

These measures confirm that when head plates are used with grouted bolts, the plates must be of the same quality and dimensions as those used with mechanically anchored bolts (Section 3.1.3).

3.2.7 Instrumentation and Grouting Quality Control

Apart from the pull-out tests described in Section 3.2.5, two other types of non-destructive instruments described below can also be used with bolts grouted with resin and cement.

3.2.7.1 Strain Gauge Instrumentation

Load cells and similar devices used with mechanically anchored bolts do not provide representative results when they are used with bolts grouted over their entire length. For the latter bolts, tension is not uniform and varies along the length of the bolt depending mainly on the degree of fracturation and the modulus of elasticity of the enclosing rock (47, 48), as well as on the proximity of a joint that intersects the drill hole.

To measure tension in various points of the bolt and verify whether it is approaching a critical value near the load at rupture of the bolt barrel, the bolts can be equipped with strain gauges (34, 49). These are mounted in a shallow groove and then covered with a protective coating of resin (Figure 34). The bolt can then be installed using normal installation equipment.

Figure 35 shows the shape of a typical tension profile obtained with a resin-grouted bolt, with maximum tension measured at the centre of the bolt.

3.2.7.2 Non-destructive Control of the Grouting

A device for the non-destructive control of bolts grouted with resin or cement has recently been introduced on the market (49). This device, called a Boltometer (Figure 36), makes it possible to verify the length of the bolt and provides an evaluation of the quality of the grouting by displaying one of four letters: A, optimal; B, decreased; C, insufficient; D, very poor.

The device contains a specially designed sensor that contains piezo-electric crystals. The sensor rests on the head of the previously flat-machined bolt, and two longitudinal and transverse elastic wave pulses are emitted. When the waves travel along the bolt, a certain amount of energy is transferred through the grout to the rock and the amplitude of the wave decreases. The waves are finally reflected at the end of the bolt and are recorded on the return trip by the sensor. If the grout surrounding the bolt barrel is continuous and of good quality, the amplitude of the reflected wave is considerably dampened, much more than if the grout is crumbly or missing. The duration of travel of the waves is also used to calculate the length of the bolt.

This device was tested in 1984 by the Ministry of Labour of Ontario (50) in 10 mines. Of 154 bolts tested, 30% were in class A, 51% in class B, and 19% in classes C and D. To visualize the meaning of these categories, it was suggested that class A corresponds to a grout applied over 100% of the length of the bolt, class B to 80%, class C to 60%, and class D to 40%.

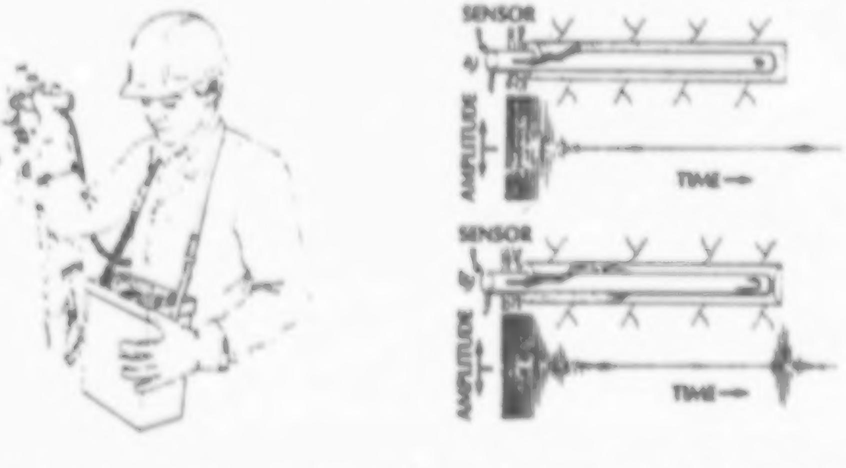


Fig. 36 – Diagram and operating principle of the Boltometer. From Thurner (49)

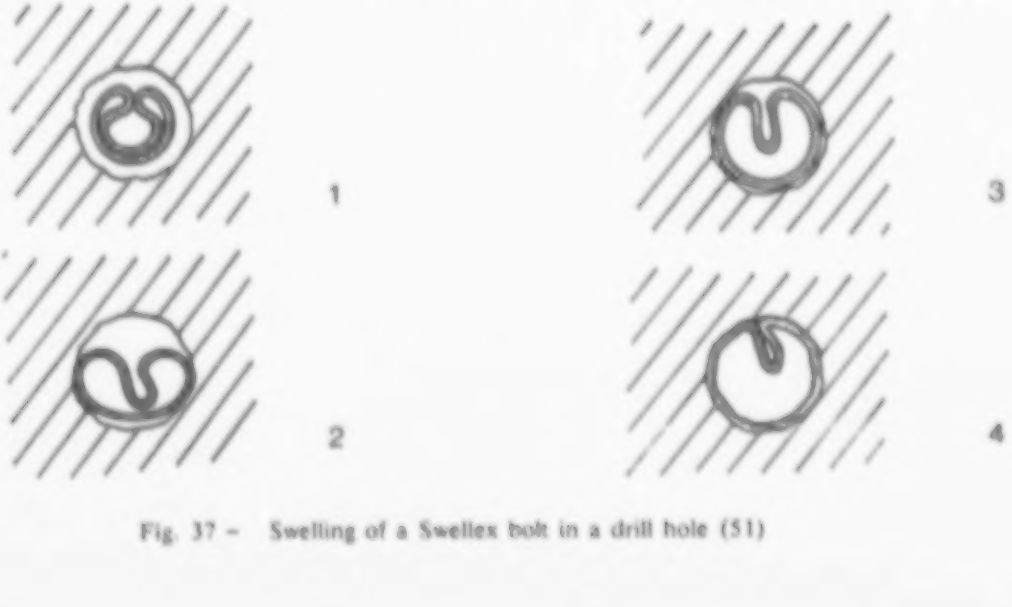


Fig. 37 – Swelling of a Swellex bolt in a drill hole (51)

3.3 FRICTION BOLTS (SWELLEX)

The Swellex (registered trademark of Atlas Copco MCT AB, Sweden) friction bolt consists of a steel tube with a diameter of 41 mm (1.61") and walls 2 mm (0.08") thick that is mechanically deformed to obtain a diameter of 25.4 mm (1") (51). The tube is inserted into a drill hole and then inflated by water pressure (Figure 37) that may vary from 20 to 30 MPa (2900 to 4350 psi), using a special pump connected to the air and water supplies of the mine.

This bolt was introduced on the market in 1982, and has several advantages:

- It requires no pushing during the installation, except for lifting the bolt.
- It has the same simplicity of installation regardless of the length of the bolt.
- The flexibility of the bolt makes it possible to install in long sections up to 9 m (30 ft) from locations with a much smaller clearance.
- It maintains contact by friction along the entire surface of the walls of the drill hole.
- The maximum holding power is mobilized from the moment of installation.
- Defective bolts can be detected by the pressure of water during the installation.
- It may be used in drill holes with diameters between 31.75 mm and 40 mm (1 1/4" and 1 9/16") (section 3.3.2).
- It has high holding power even with very short anchoring lengths, except in excessively soft grounds, where the holding power is lower, although the bolt can still be used in these latter situations (section 3.3.3).

3.3.1 Properties of the Bolt Barrels

The steel used for the bolts is soft steel with the following properties (20):

Yield strength: not available
Ultimate strength: 56,500 psi (390 MPa)
Load exceeding the yield strength: not available
Load at rupture: 22,502 lb (100 kN)
Percentage of elongation at rupture: 7% over 200 mm

Because of their tubular section, the bolts are particularly sensitive to corrosion, especially on their external surfaces. Various anti-corrosion treatments can be applied in the plant, such as galvanizing, or certain hot treatments that can be as effective and are less expensive.

3.3.2 Holding Power and Minimum Anchoring Length

One characteristic of Swellex bolts is their variable holding power. It fluctuates not only as a function of the type of rock where they are installed, but also as a function of the characteristics of the installation, that is, the diameter of the drill hole and the inflation pressure.

Figure 38, established by the manufacturer of Swellex bolts, illustrates the effect of these two parameters on the holding power per metre of bolt installed in hard rock (51). The strength may vary by a factor of two, from 100 to 200 kN/m, for each of these two parameters, and would vary by a factor of four, from 50 to 200 kN/m, if we take into account the two extremes: a bolt installed in a 31.75 mm (1-1/4") hole with a pressure of 20 MPa, and a bolt installed in a 38 mm (1-1/2") hole with a pressure of 30 MPa.

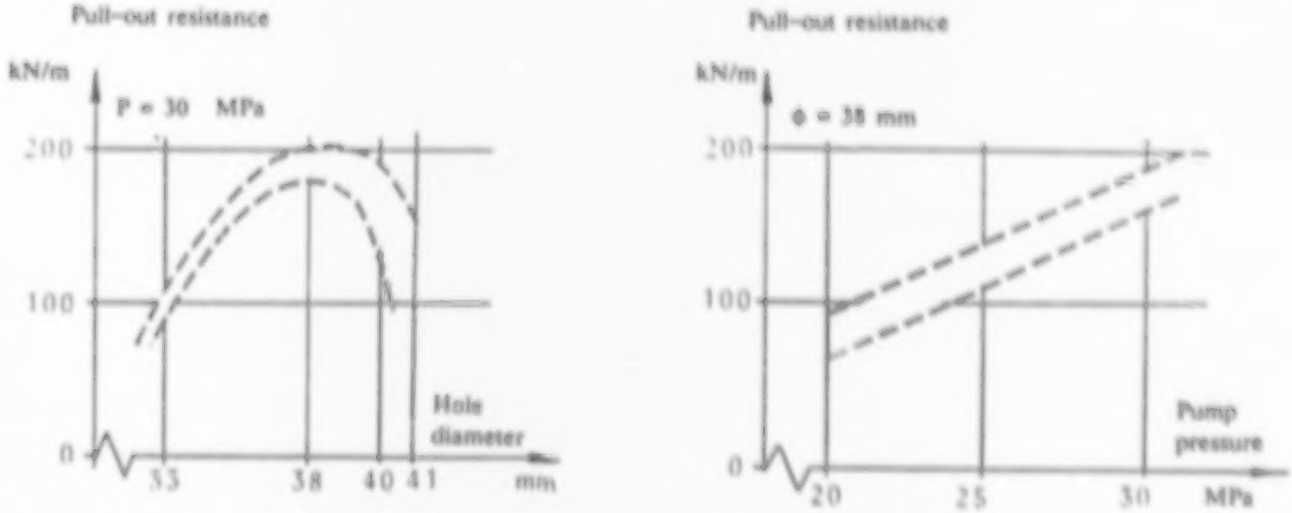


Fig. 38 - Pull-out resistance of Swellex bolts as a function of the drill hole diameter and the inflation pressure (51)

The two graphs make it possible to evaluate the minimum required anchoring length for Swellex bolts (Figure 39) to obtain an anchoring capacity at least equal to the load at rupture of the bolt. For example, under the best conditions, the minimum anchoring length (L) can be obtained using the following formula.

$$L = \frac{\text{load at rupture}}{\text{pull-out resistance}}$$

$$= \frac{100 \text{ kN}}{200 \text{ kN/m}}$$

$$L = 0.5 \text{ m}$$

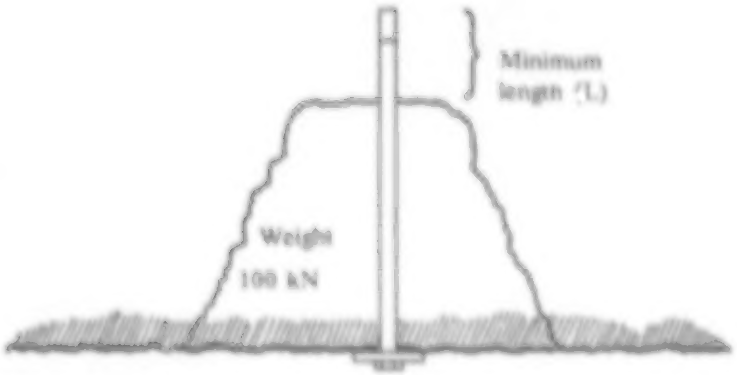


Fig. 39 - Illustration of the minimum anchoring length (L) of a Swellex bolt

This minimum length is similar to the figure of 0.36 m obtained in Section 3.2.5.2 for a cement-grouted 19 mm (3/4") bolt (load at rupture of 118.9 kN (26,720 lb)) in a 38 mm (1-1/2") drill hole. For the same 19 mm (3/4") bolt grouted with resin in a 25.4 mm (1") drill hole, the minimum grouting length obtained from the table in Figure 30 would be 0.63 m.

Nevertheless, since the pull-out resistance of Swellex bolts can be as low as 50 kN/m (or less in certain cases), it is important to carry out pull-out tests in each site, to confirm the value that must be taken into account in the calculations. The equipment used to carry out the pull-out test is shown in Figure 40.

Since the bolt was only recently introduced, and most tests have been carried out in hard rock, pull-out tests in soft rock (compressive strength under 40 MPa) are necessary to draw graphs that can be used to select pull-out resistance similar to those shown in Figure 38. The partial results discussed in the next section show that Swellex bolts retain a relatively high holding power in soft rock as well.

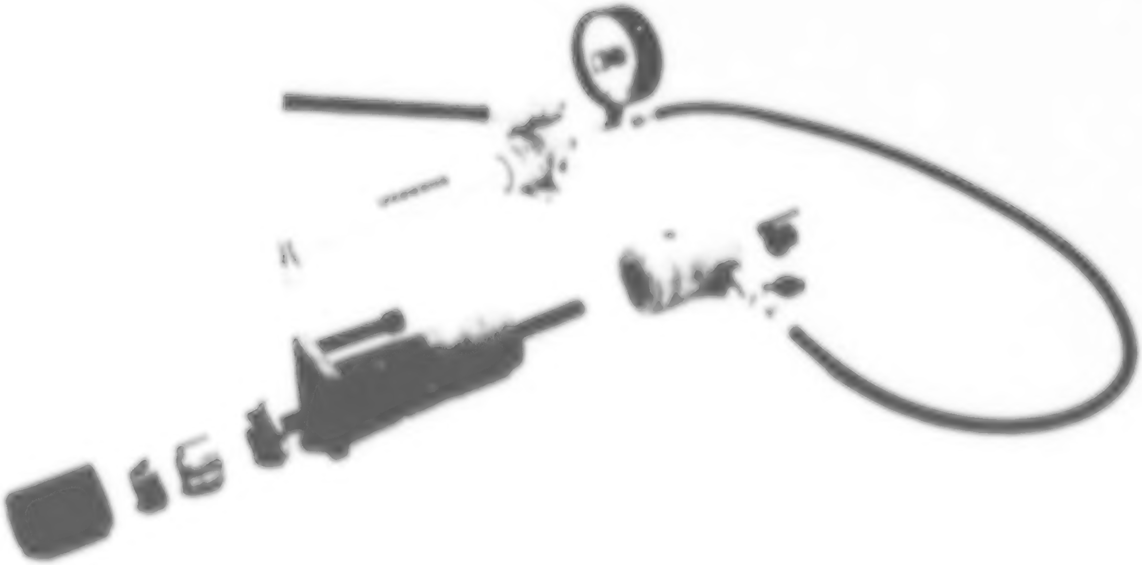


Fig. 40 - Device used to pull out Swellex bolts (51)

3.3.3 Utilization Area

Tests using Swellex bolts have been carried out in most types of grounds (soft and hard sedimentary rock, volcanic rock, intrusive rock: granite, etc.) and have produced satisfactory results for most of them according to the manufacturer (51), in that the pull-out values obtained were high, often near the load at rupture of the bolt.

For soft rocks, the results of pull-out tests discussed in Appendix 16 show widely variable unit slip loads, from 30 or 50 kN/m to values almost as high as those observed in hard rock.

Nevertheless, if we take the example of section 3.3.2 on the calculation of the minimum anchoring length of the bolt with the above value of 50 kN/m, the minimum length obtained would be 2 m. In many cases this value is excessive because we must take into account that it is the anchoring length of the bolt beyond the fissure or block that we want to keep in place. In such situations of bolting soft rocks, the use of Swellex bolts should be evaluated by comparing them with the anchoring capacity that can be obtained with bolts grouted with resin or cement (Sections 3.2.4 and 3.2.5).

The results discussed in Appendix 16 also include tests carried out in a stiff clay where unit slip loads obtained were relatively high (16 to 37 kN/m), something that could probably not have been possible with any other type of support.

3.3.4 Accessories and Special Uses

3.3.4.1 Using a Steel Tube

For applications where we want to increase considerably the elongation capacity of the bolt, it is possible to insert a steel tube near the head of the bolt (Figure 41). This tube prevents the bolt from inflating, and this ensures a greater elongation distance between the head plate and the friction area where the bolt is inflated. Such tubes are used in mine 9 (16) in sub-level galleries where, in the past, the significant amount of convergence in the sericitic schist walls used to cause the rupture of the bolts because of their low elongation capacity.

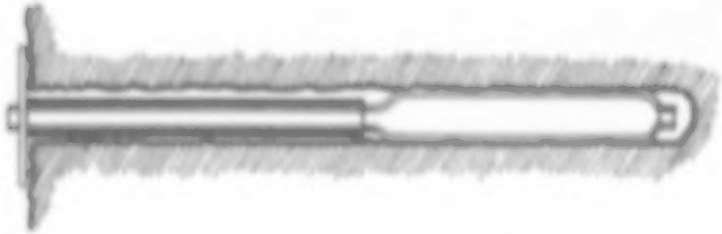


Fig. 41 - Insertion of a steel tube near the head of a Swellex bolt

The inflation of the Swellex bolt also involves a slight shrinkage of its length that helps to tighten the head plate against the ground. In very crumbly grounds, this tightening of the plate may loosen rock fragments. To counteract this effect, the use of short 15 cm (6") pipes is recommended.

3.3.4.2 Installation of a Wire Mesh

A special plate (Figure 42) has been developed to make it possible to install a wire mesh quickly over Swellex bolts. This plate has a diameter of 10 cm (4") and is inserted over the head of the bolt by hammering. Tests have shown that the holding power of the plate is 17.8 kN (4,000 lb). This value is comparable to the load-bearing capacities of mine wire meshes (Section 3.1.7).

3.3.4.3 Installation of Bolts Longer than the Gallery Clearance

Because of their flexibility, long Swellex bolts can be installed in galleries with restricted clearance. The installation procedure is shown in Figure 43. In many cases, we could even use very long Swellex bolts to replace the cable bolts to control dilution in underground mining stopes.

For example, a test carried out by the manufacturer (53) has shown that it is possible to install 6 m (19.7 ft) bolts in a gallery measuring 2.4 m (8 ft)in width and 2.7 m (9 ft)in height. Moreover, by inclinating the drill holes by 15° in relation to a line perpendicular to the wall, we could expect to increase the length of the bolts installed even more.

3.3.4.4 Anchoring a Cable in a Drill Hole

The Swellex bolt can be used to anchor a cable with a diameter of 9.5 mm (3/8") in a 44.5 mm (1 3/4") hole (54). An interesting application is the lacing technique that is presently being tested in Ontario in galleries susceptible to rock bursts. A row of cable is reeled out, and the cable is inserted at regular intervals into 1.5 m (5 ft) long drill holes with a Swellex bolt which is then inflated(Figure 44). This technique can also be used to reinforce draw-points.

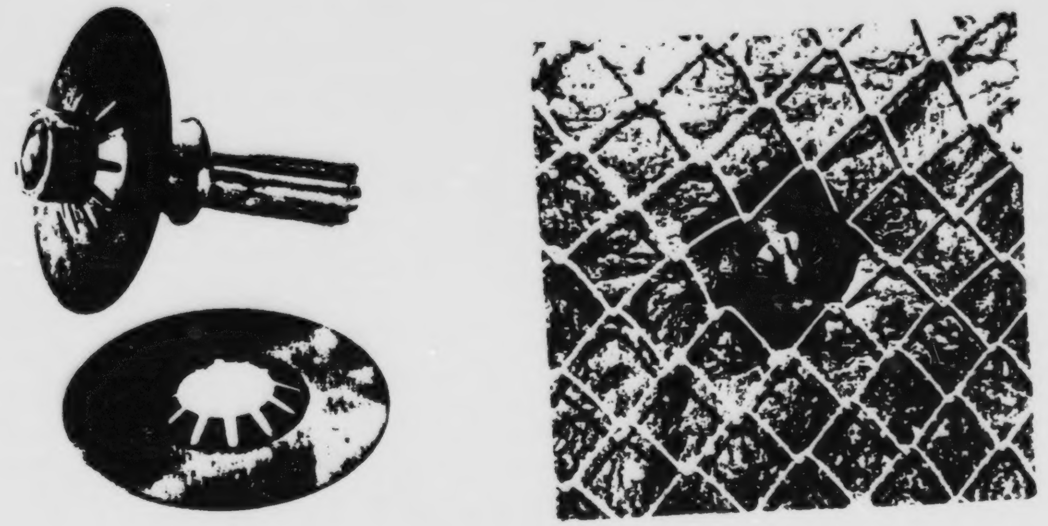


Fig. 42 – Plate used to attach the wire mesh to the head of a Swellex bolt

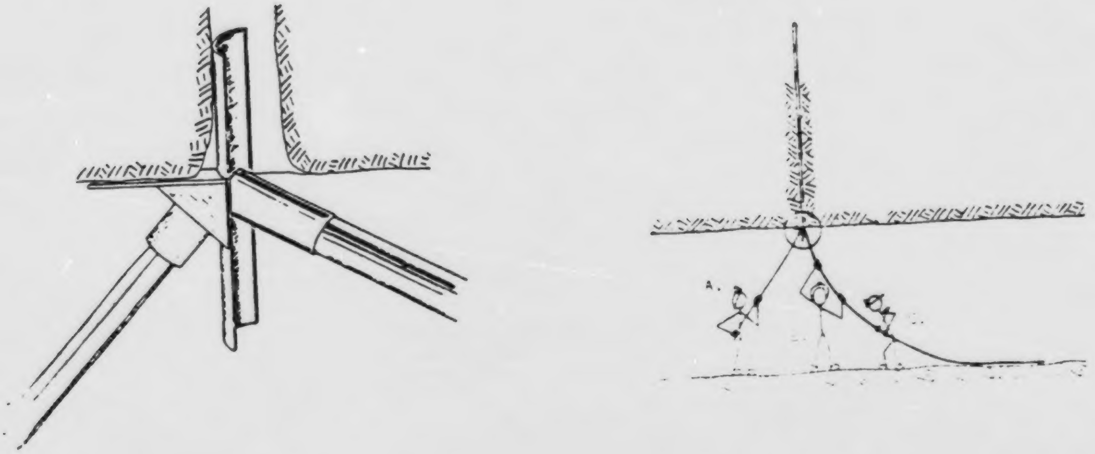


Fig. 43 – Method used to install very long Swellex bolts (53)

3.3.4.5 Hydraulic Tappet

A device known as a water leg can be used with the standard equipment to install Swellex bolts. This stands on the floor of the gallery and makes it possible to install bolts under the most difficult conditions. Some of these conditions include inserting very long bolts in narrow drill holes, applying enough thrust to stretch the wire mesh during the installation, ensuring good contact with the head plate, and lifting heavy objects such as steel beams.

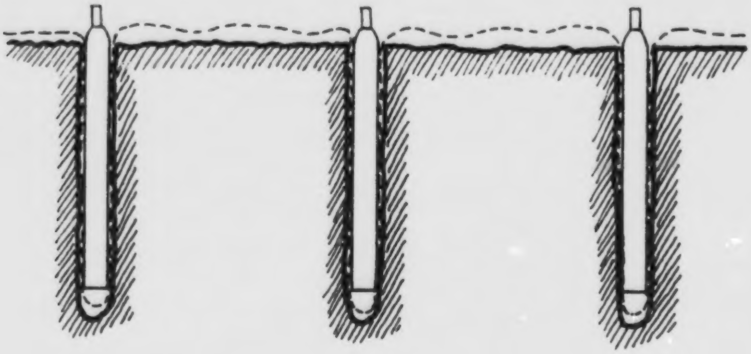


Fig. 44 – Anchoring a cable in a drill hole (lacing technique)

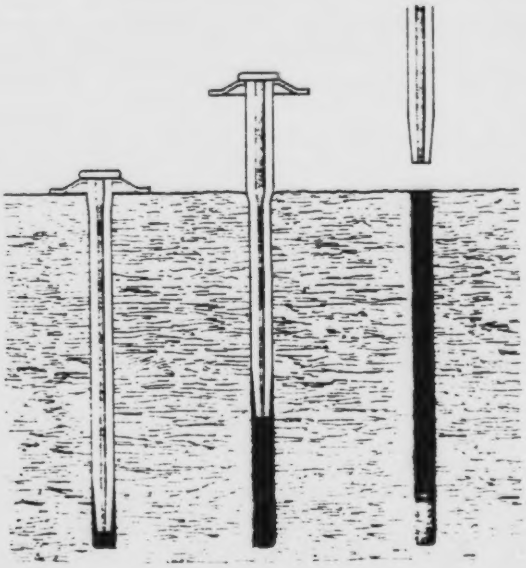


Fig. 45 – Split Set friction bolt

3.4 FRICTION BOLTS (SPLIT SET)

The Split Set (registered trademark of Ingersoll Rand Co., U.S.A.) friction bolt was invented in 1973 (55) and has been on the market since 1978.

It consists of a steel tube with a slit that runs lengthwise. It has been designed to be inserted in a drill hole with a slightly smaller diameter (Figure 45).

3.4.1 Properties of the Bolt Barrels

The bolt barrels are made from steel sheets that are shaped in the form of a tube. One of the ends is tapered, and a welded ring is added to the other end to hold the head plate. The holding power of the ring is rated at 102 kN (23,000 lb) by the manufacturer.

There are three bolts which are suitable for three different drill hole diameters.

Bolt	Diameter of the tube (in)		Diameter of the drill hole (in)	Thickness of the wall of the tube (mm)	Length of the tube
	min	max			
SS-33	1.30	1.36	1 1/4	2.2	up to 2.4 m
SS-39	1.50	1.56	1 3/8	2.2	up to 3.0 m
SS-46	1.78	1.84	1 5/8	3	1.5 to 3.7 m

Bolt SS-33 is the most frequently used in Canada, because it is customary to use drill holes with a diameter of 31.75 mm (1 1/4").

The bolts are made of high strength steel with the following properties:

Yield strength: 60,000 psi (414 MPa)
Ultimate strength: 75,00 psi (520 MPa)
Load exceeding the yield strength (SS-33): 18,800 lb (83 kN)
Load exceeding the yield strength (SS-39): 21,400 lb (95 kN)
Load at rupture (SS-33): 23,500 lb (104 kN)
Load at rupture (SS-39): 26,800 lb (119 kN)
Percentage of elongation at rupture: 16% over 200 mm

The bolts may also undergo an anticorrosion treatment by galvanization.

3.4.2 Insertion Method

The bolts are inserted into the drill holes with a percussion drill. The machine may be a portable drill (equipped with a jackleg) or a specially adapted drilling jumbo, which feeds the bolts on the drill feed. Some bolting jumbos are also adapted so that they can be used to install Split Set bolts.

Various adaptors that can be used for insertion using portable drills are available, including an offset tool that facilitates insertion in the roof of galleries that are not very high. Instead of using this tool for these lower galleries (about 2.5 m), however, we recommend drilling the first 50 cm in a larger diameter (which would allow the bolt to be inserted freely), drilling the rest of the hole with the diameter for which the bolt was designed, and then inserting the bolt with the jackleg drill (Figure 46).

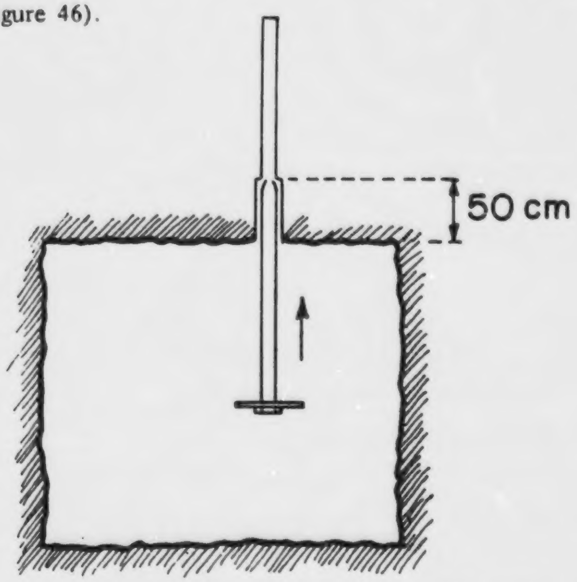


Fig. 46 – Insertion of Split Set bolts in small galleries

Although the most commonly used Split Set bolt in Canadian mines is the SS–33 model, the SS–39 bolt is generally used in American mines. Many Canadian users have pointed out that they have encountered certain insertion difficulties with SS–33 bolts longer than 1.5 m (5 ft), and that they have, in practice, used bolts of these or shorter lengths. The problem in this case is probably the diameter of the drill holes, for which the manufacturer suggests a tolerance of +2.5 mm (+3/32") and –1.6 mm (–1/16"). Normal wear of the drill bit may be such that this –1.6 mm (–1/16") tolerance is quickly exceeded, and a smaller diameter drill hole makes it more difficult to insert the bolt.

On the other hand, certain mines in soft rock, particularly in the United States, have not reported any difficulties with the insertion of the SS-39 bolts, even when they use bolts 1.8 m (6 ft) or 2.5 m (8 ft) long.

3.4.3 Holding Power

Split Set bolts are generally reputed to have a lower holding power in comparison with other types of bolts, particularly when the pull-out tests are carried out immediately after installation. Holding power subsequently increases over time. This is owing to corrosion on the surface of the bolt barrels, which improves the friction characteristics, and to any ground displacements that may help wedge the bolts into the drill holes.

For holding power immediately after installation, the main parameter that must be met is the diameter of the drill hole (56), as shown in Figure 47. This figure, which was established by the manufacturer for SS-39 bolts, shows that the holding power drops by a factor of about 2.5 when the diameter of the drill hole exceeds by more than 2.5 mm (3/32") the nominal diameter of 35 mm (1 3/8"). This tolerance in excess of the nominal drill hole diameter is about the same as that allowed for mechanically anchored bolts (Section 3.1.9), and Swellex bolts (Section 3.3.2). The same tolerance of +2.5 mm (+3/32") probably applies also for the SS-33 bolts.

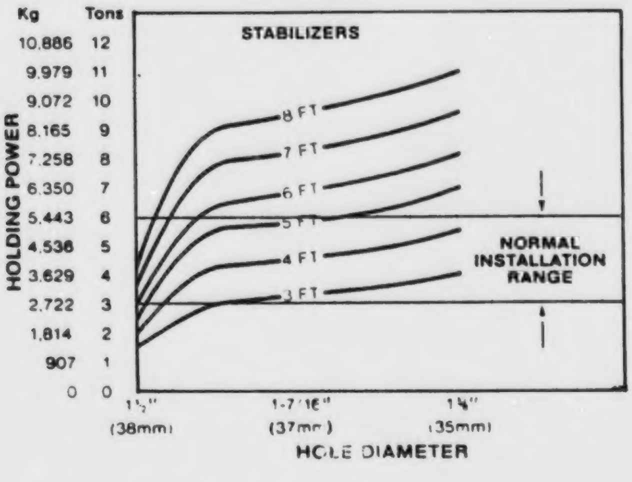


Fig. 47 – Holding power at installation of SS-39 Split Set bolts as a function of the drill hole diameter (56)

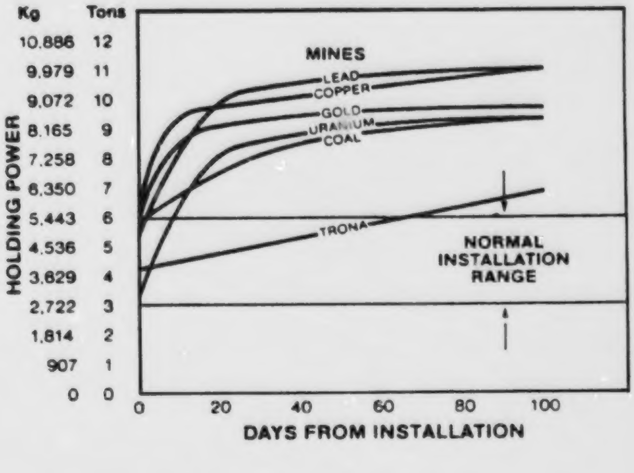


Fig. 48 – Holding power of Split Set bolts depending on the nature of the enclosing rock and the time elapsed since installation (56)

The increase in holding power as a function of time is shown in Figure 48 for various types of rock, from soft to medium, found mainly in American mines. The figure suggests that the holding power is about the same for all rocks when they exceed a certain strength threshold. But we still have very little information on the long-term holding power of Split Set bolts; pull-out tests should be carried out on any new site where they will be used on a large scale. Canadian sites in hard rock should receive special attention, since very few tests have been carried out until now in such locations.

Results of some pull-out tests using Split Set bolts carried out by various independent observers are reported in the literature. In particular, Myrvang *et al.* (56) obtained a pull-out load of 51 kN (11,500 lb) with a 1.7 m (5.6 ft) bolt installed in a schistous rock; Singh *et al.* (58) obtained about the same pull-out load, 52 kN (11,700 lb). These two tests were probably carried out on SS-39 bolts.

3.4.4 Utilization Area

The users of Split Set bolts contend that they differ from other types of bolts in that they allow a certain amount of slip.

This slip takes place when the bolt is overloaded (Figures 47 and 48) and to the extent that ground displacements have not produced excessive wedging, in which case the bolt will quickly exceed its rupture load.

This ability to slip should thus be considered an advantage (59), because it allows the bolt to adapt to various types of ground while maintaining a certain level of holding power; other bolts would reach rupture under identical conditions because of their higher stiffness.

We may then consider that Split Set bolts are particularly suitable for grounds where large displacements are likely. Very often, these grounds are relatively soft, sedimentary or igneous schists, or are made up of sedimentary rocks, such as sandstone and porous limestone.

At present, Split Set bolts are used in many metallic and non-metallic mines in the United States (59), especially in uranium mines, where they are the main type of support used. These mines are in excessively soft sedimentary formations, that often have a compressive strength of less than 3 MPa (500 psi). This means that timber bracing had to be used prior to the introduction of Split Set bolts. In these formations, mechanically anchored bolts and resin-grouted bolts cannot be used, since the walls of the drill hole crumble when the bolt is rotated to mix the resin.

At this time, Split Set bolts are not used in American coal mines, which nevertheless represent the largest market for rock bolts. Many of these mines exploit the coal by the room-and-pillar method and suffer relatively serious ground problems because of the heavily bedded formations found in the roof of the galleries. At present, the bolts used are mechanically anchored and resin-grouted bolts, as well as combination bolts (Section 3.2.1.1). In some cases, Split Set bolts may nevertheless offer satisfactory performance characteristics in bedded formations in the roof of galleries, and this has been confirmed by the results of the studies discussed in Appendix 17.

Another area of application for Split Set bolts seems to be in galleries where rock bursts are likely to occur. Scott (55) mentions the galleries in mines in the Coeur d'Alène district in Idaho, where the exploitation reaches depths of 2,400 m (7870 ft) in very hard quartzites. Some of these galleries measuring 2.7 x 3 m (9 x 10 ft) are supported by Split Set bolts placed in 0.9 x 1.2 m (3 x 4 ft) centres using a no. 16:1.58 mm (0.063") gauge wire mesh. When rock bursts of medium intensity occur in these galleries, the rock enclosing the gallery fractures but is kept in place by the support. Subsequently, restoration efforts are enough to render the gallery usable again. In these same mines, a rock burst measuring 2.0 on the Richter scale in a stope exploited by the shrinkage mining method caused no damage in the central section of the stope which was supported by Split Set bolts while the timbered ends of the same stope were lost.

In the same way, Hedley *et al.* (60) report the satisfactory performances of a support system that withstood rock bursts in a uranium mine in the Elliot Lake region. The support consisted of resin-grouted bolts, wire mesh, and Split Set bolts.

Finally, some users of Split Set bolts have observed the breakage of the welded ring at the end of the barrel (16, 59). This problem develops when the slip mentioned above is prematurely blocked by displacements in the ground. This rupture must be interpreted as a warning of the need to reinforce the support or to replace it with another bolt that is more compatible with the expected deformations.

Chapter 4: SELECTION OF ROCK BOLTING CHARACTERISTICS

The bolting characteristics chosen are the following:

- type of bolt;
- length and spacing of the bolts in the gallery.

This chapter suggests several methods for selecting the type of bolt based on the quality of the grounds or on a scale expressing the stiffness of different bolts.

The chapter also suggests field methods to evaluate the main geomechanics classifications that can be used to determine whether bolting is necessary in the gallery or stope under consideration. If bolting is required, the length and spacing of the bolts can be calculated using the Norwegian Geotechnical Institute (NGI) classification. At the end of the chapter, this classification is compared with a few other classifications and empirical rules that can be used to choose the length and spacing of the bolts. The comparison shows that the NGI classification is the most realistic method to choose the length and spacing of bolts.

Chapter 4:
SELECTION OF ROCK BOLTING CHARACTERISTICS

4.1 INTRODUCTION

The selection of rock bolting characteristics involves two stages: the choice of the type of bolt and the choice of the density of rock bolting, that is, the number of bolts installed per square metre of wall.

Initially, we must understand that these two parameters are usually chosen based on experience acquired in the mine. Nevertheless, there are many suggestions in the literature that can be used to arrive at a rational choice of rock bolting characteristics. At present, the main contributions can be found in the geomechanics ground classifications established by various authors who have surveyed a large number of underground installations. These authors have analysed the support used, as well as the prevailing ground conditions there. The main geomechanics classifications in current use are discussed below.

There is a great need to find methods to arrive at a choice of support; however, we must recognize that the methods developed until now have been applied only to a very limited extent to mining galleries and stopes. One of the main reasons may be that the geomechanics classifications have been developed based on analyses of civil engineering installations (tunnels, hydroelectric power plants), for which support has traditionally been heavier than for mining installations, particularly because the former are permanent constructions.

Part of the work undertaken to prepare the *Rock Bolting Practical Guide* involved a survey of the rock bolting characteristics of 13 Quebec mines carried out in 1984 (16). This survey made it possible to show that the density of rock bolting used in the galleries is only partly determined by the quality of the ground. In practice, it happens to be more a function of two other parameters: the width and the depth of the galleries. The results of this survey are discussed in Appendix 18. When we consider the length of galleries opened per year in the

13 mines and the total number of bolts used during the same period, the practice which we encountered on average in the mines is the following:

Rock bolting conditions	Light	Intermediate	Normal	Reinforced
Rock bolt per metre square	0.2	0.6	1.0	1.5
Equivalent bolt pattern	2.25 x 2.25 m	1.3 x 1.3 m	1 x 1 m	0.8 x 0.8 m

The rock bolting conditions are defined as follows (see Appendix 18):

Rock bolting conditions	Light	Intermediate (first type)	Intermediate (second type)	Normal (first type)	Normal (second)	Reinforced
Depth	30–175 m	175–500 m	30–175 m	175–500 m	500–750 m	500–750 m
Width of the galleries	2.7 m	2.7 m	4–5 m	4–5 m	2.7 m	4–5 m

Unless we accept that ground conditions are identical at the same depth in all Quebec mines, something that is very unlikely in view of the different geological contexts, the results of this survey confirm that, at this time, the usual practice is to progress toward a heavier type of support as depth increases. Although the support is made locally to suit the conditions of the ground, the general tendency which we have observed is an increase in reinforcement with depth.

This increase is, in our view, not always justified, as shown by the graph in Figure 49 illustrating the densities of fracturation as a function of depth observed in 57 gallery sections distributed among the 10 mines visited during the course of 1985 (18). Even though the sample was relatively small, it is interesting to note that the density of fracturation seems to decrease with depth down to 350 m, and then starts to increase again.

This chapter on the selection of rock bolting characteristics takes into account the above-mentioned observations and provides an overview of the methods and procedures for choosing the support as a function of the ground quality. The methods proposed are based on the principle of using geomechanics classifications adapted for underground mines.

Many studies were carried out in parallel with the preparation of this guide (17),to test how the methods proposed in Section 4.2 can facilitate the investigation of ground quality. Thus, these methods make it easier to use the geomechanics classifications to select rock bolting characteristics.

4.2 METHODS OF GROUND CONDITION ASSESSMENT

The main parameters to determine the quality of the ground are:

- unconfined compressive strength of the rock (σ_c).
- rock quality designation (RQD), or more generally, the average spacing of the joints and other geological discontinuities in the rock;
- degree of alteration on the surface of the joints;
- in-flow of underground water;
- the number of joint sets and average spacing of the joints in each set;
- joint strike and dip orientations that would tend to facilitate the fall of rock blocks;
- the size of joints that mark the boundaries of rock blocks of significant volume;
- stress conditions of the ground in the vicinity of the gallery or underground opening, proportional to its depth (see Appendix 3);
- predicted variation in the stress conditions, for example, those caused by the exploitation of an adjacent stope;
- blasting vibrations.

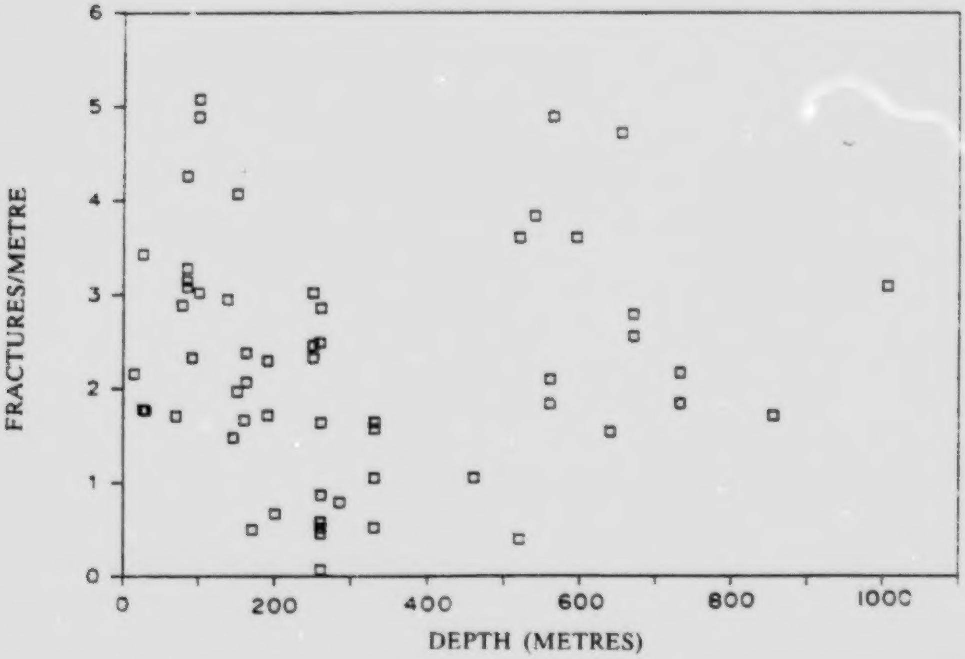


Fig. 49 - Relationship between density of fracturation and depth in 57 gallery sections. From Charette *et al.* (18)

These parameters are generally evaluated globally by miners and their supervisors when the time comes to evaluate the quality of the ground and to choose a suitable support on the site itself.

The difficulty in introducing rational methods to evaluate the quality of the ground arises because, of all the parameters mentioned, only the first two, and to a lesser extent the fifth, sixth, and seventh, lend themselves to an evaluation based on quantitative methods. All other parameters are qualitative and can only be evaluated in relation to other situations experienced by the observer or others.

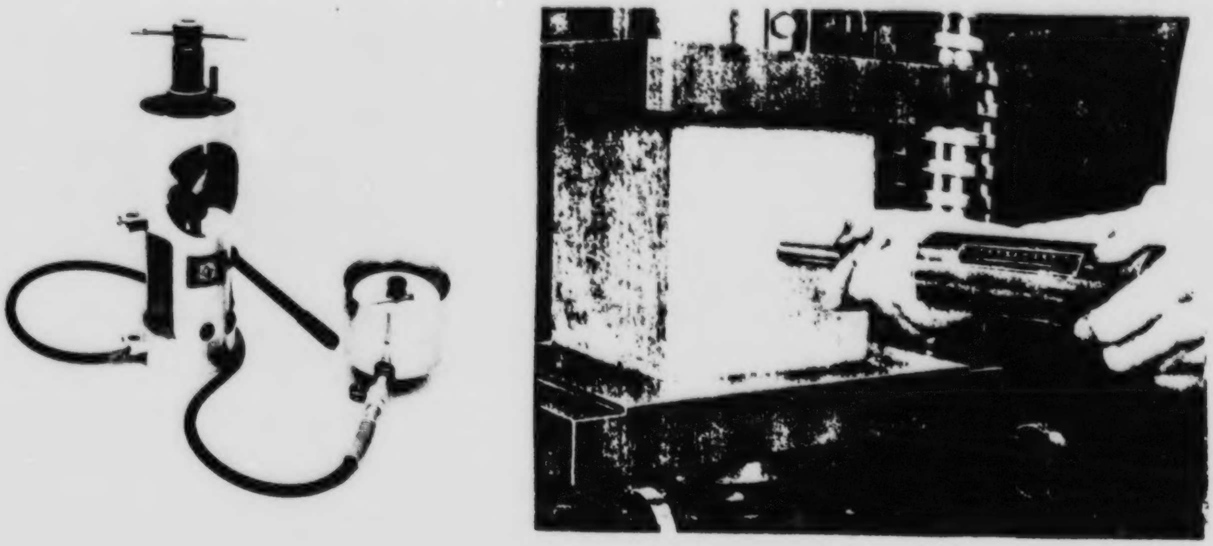
All the geomechanics classifications discussed in Section 4.2.3 have in common the use of the first four parameters mentioned. Some of them also use one or several of the other parameters, and this tends to determine a classification's particular purpose, which is also discussed in the same section.

The evaluation methods that can be used in the field to evaluate compressive strength and RQD, and to identify the main joint sets are discussed in the following sections.

4.2.1 Evaluation of Compressive Strength in the Field

The compressive strength of rock is a reference parameter that is very often used to quantify the hardness of a rock. The method used to determine this property in the laboratory has been standardized by the ASTM (Standard–D2938: Unconfined Compressive Strength of Intact Rock Core Specimen). This test is carried out on core samples of rock with an NX diameter (54 mm), whose parallel ends have been ground to within a very low tolerance.

At the same time, field methods have been developed that make it possible to estimate this same parameter with less precision, but that have the advantage of being fast and providing immediate results. The two principal methods shown in Figure 50 are the point load and Schmidt hammer tests.



a) Point load tester

b) Schmidt hammer

Fig. 50 – Devices used to determine the compressive strength of rock in the field

4.2.1.1 Point Load Test

This test may be carried out using core samples of rock or rock pieces of irregular shape and diameters from 20 mm to 60 mm (0.8 to 2.4"). It consists of loading the sample to rupture and then determining the point load index I as follows:

$$I = \frac{P}{D^2}$$

where P: load at rupture (lb or MN)

D: diameter (in or m)

I: in lb/in^2 or MPa

Compressive strength in lb/in^2 or MPa is then evaluated by multiplying index I by a factor shown below.

Diameter of the sample (mm)	20	30	40	50	54	60
Factor	17.5	19	21	23	24	24.5

Nevertheless, it must be emphasized that the factors suggested have been mainly determined on the basis of tests carried out on sedimentary rocks. Several users of the test on rocks in the Canadian Shield have pointed out that the factor that should be used is very often lower than those listed above. Consequently, when introducing the testing method in a particular site, several compressive strength tests should be carried out in the laboratory to calibrate the method.

4.2.1.2 Schmidt Hammer

The Schmidt hammer is a device that was initially developed in Switzerland to obtain a quick determination of the compressive strength of concrete cylinders under the conditions of the work site. Several studies have been carried out by investigators to adapt the method for use with rocks, either with rock core samples or for direct use on the site, by applying the hammer directly against the wall of the gallery or stope under consideration.

This device has the advantage of being portable and easy to use. The procedure to be followed when using it in a gallery is described in detail in Appendix 19 and summarized below.

a) Find the location to be tested. It should preferably consist of a straight and vertical surface. It is possible to carry out the test on a horizontal surface on the roof of a gallery or at a 45° slope, but the formula shown below has to be modified.

b) Clean the surface to remove any traces of rock dust.

c) Apply the Schmidt hammer and repeat the test 20 times, moving the hammer at least 1 cm after each impact.

d) Obtain the average of the 10 highest readings (rebound index R).

The formula used to evaluate the uniaxial compressive strength σ_c valid for a type L hammer held vertically down is shown below:

$$\sigma_c = 6.9 \times 10^{\left(0.16 + 0.0087 \cdot (R.\gamma - 7.5)\right)}$$

where R: rebound index

γ: density of the rock (g/cm^3)

σ_c: uniaxial compressive strength of the rock (MPa)

The validity of this formula has been confirmed by tests discussed in Appendix 19 which were carried out on rocks of the Canadian Shield. When the hammer is held horizontally or inclined at a 45° angle, or when the density of the rock is higher than 3.2, it is necessary to use another formula, shown in Appendix 19.

4.2.2 Evaluation of RQD in the Field

By definition, RQD is determined using boxes of core samples. It is defined as follows:

$$\text{RQD} = \frac{\text{Sum of sample lengths over N cm}}{\text{Total length of drilling}}$$

where N is a length that varies depending upon the diameter of the rock core samples. At the beginning, Deere (61), who introduced the RQD concept, suggested that it should be determined using rock core samples with an NX diameter (54 mm, 2 1/8") obtained with a double core barrel drill. This would ensure that there was minimum shaking of the samples and that the fractures observed could all be considered to be natural. In this case, the author suggested making N equal to 10 cm.

However, the use of the RQD has been extended to include core samples of smaller diameter, particularly in the mining sector where it is rare to find drill holes with an NX diameter. With smaller drill holes, it is even more important to ensure that the drilling is carried out under conditions that minimize shocks. When it is possible to comply with this condition, the values of N to use as a function of the diameter of the core sample are shown below.

N (cm)	6.3	7	8.9	9.6	10
Diameter of the core sample (mm)	20.6 (EX)	27 (AQ)	41 (BX)	47.6 (NQ)	50 or more

Determining the RQD of a rock can be considered as a routine exploration task to be entered into the drilling logbook in the same way as other normally surveyed parameters, such as the percentage of recovery, the drilling speed, the colour of the wash water, etc.

The value of the RQD already provides an indication of the degree of fracturation and of the quality of the ground. Thus, Deere (61) proposes the following classification:

RQD	Quality of the ground
< 25%	Very poor
25 - 50%	Poor
50 - 75%	Average
75 - 90%	Good
90 - 100%	Very good

This classification shows clearly that the support of grounds with an RQD of 70% may already be affected by considerable problems. This demonstrates the importance of determining the parameter with an acceptable degree of precision.

On the other hand, the use of the RQD parameter alone to classify the ground is not always enough. For example, there are reports of grounds that contain relatively widely spaced fracture networks, with an average spacing between fractures of 0.38 m (1.25 ft), and an RQD of 95%, in which support nevertheless presents considerable problems (63). This observation confirms the need to use the geomechanics classifications discussed in Section 4.3, which use RQD to evaluate the properties of the grounds.

4.2.2.1 Estimating the RQD by Counting Fractures in a Gallery

One major disadvantage that limits the use of RQD in practice is the fact that it may be determined very often only on boxes of rock core samples that have been obtained away from the precise location where we want to evaluate the ground conditions. The method described below makes it possible to get around this difficulty. It is based on the relationship, observed by several authors (64, 65), that exists between the number of fractures per metre in a gallery and the RQD of rock core samples obtained from the same location.

The most commonly used formula for this purpose is the following (65):

$$RQD = 100.\ (0.1\lambda + 1).e^{-0.1\lambda}$$

where λ : number of fractures per metre

The validity of this formula for igneous rocks in the Canadian Shield was established on the basis of surveys carried out in 10 mines visited during the course of 1985 (18). The studies that were carried out are described in Appendix 20.

Thus, the following procedure can be used to determine the RQD by counting fractures in a gallery.

a) Select a portion of gallery (or stope) measuring 15 m(50 ft) in length, which is representative of the grounds that are to be evaluated.

b) Establish a traverse by stretching a 15 m (50 ft) tape measure to the wall.

c) Count the number of fractures 30 cm (1 ft)long or more that intercept the tape measure. The fractures that have been clearly caused by blasting must also be counted, because they have a local effect on the quality of the ground.

d) Establish the number of fractures per metre, λ, using the following formula:

$$\lambda = \frac{\text{number of fractures 30 cm (1ft) or longer}}{\text{15 m}}$$

e) Calculate the RQD with the formula shown above.

It is important to point out that the use of the formula requires a critical attitude on the part of the user. Depending on the natural orientation of the fractures in the ground, which may be near the horizontal, subvertical, or mostly oriented in a preferential direction, the value of the RQD will depend on the direction of the survey. All this is equally true in the case of an RQD calculated on the basis of rock core samples. Thus, sometimes, it will be necessary to count fractures in different directions to obtain a representative average value of RQD.

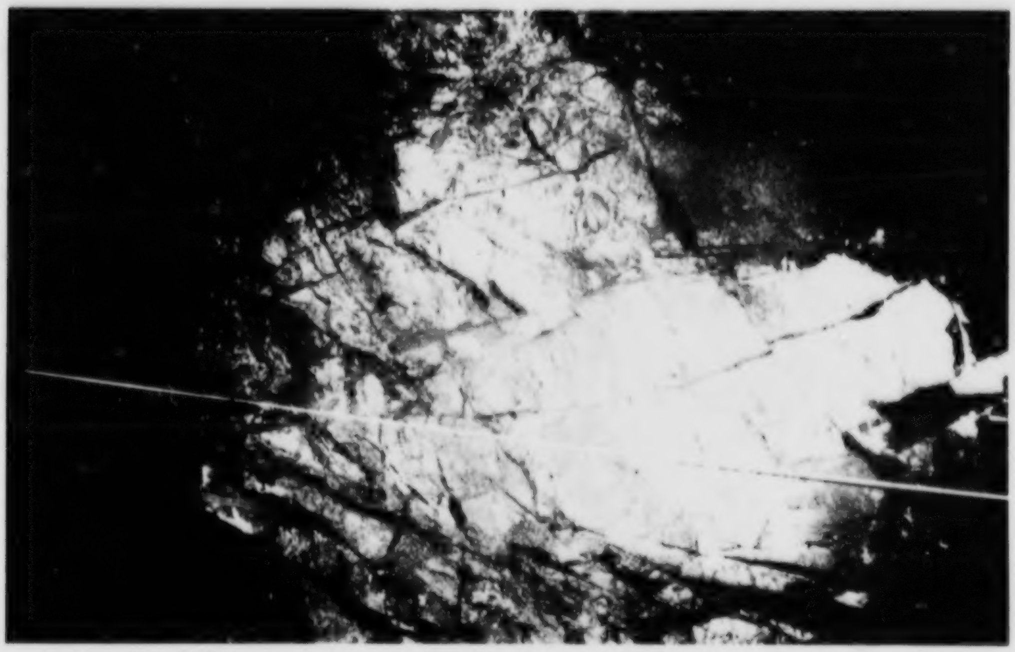
4.2.3 Identification of the Main Discontinuity Sets

We may consider that the ground conditions will be radically different depending on whether we find one or more (up to four or five) discontinuity sets in a given site. Some geomechanics classifications take this into account.

Depending on the nature of the ground, the identification of the main discontinuity sets can be obtained very quickly while counting fractures along a 15 m traverse as described in the previous section; in other cases, however, a complete discontinuity survey using a compass and subsequent plotting on a stereographic projection may be required.

The simplest cases are those where there are one or two sets of discontinuities, for example, one schistosity with very narrow spacing, and one joint set with wider spacing that is not parallel to the schistosity, or any other situation where it is easy to discriminate the discontinuities visually so that they can be classified into distinct sets.

We strongly recommend that an attempt be made first to classify visually the discontinuities present along the traverse, as suggested in the previous section, for the determination of RQD. The user can thus quickly acquire a good understanding of the type of ground with which he or she often deals, and can then arrive quickly at a classification. As an example of the visual classification we are suggesting, the photographs shown in Figure 51 illustrate different types of grounds encountered during the surveys carried out in 10 Quebec mines, the results of which are discussed in Appendix 21. Each photograph indicates the number of the corresponding gallery, the number of fractures per metre surveyed in accordance with the method described in Section 4.2.2.1, and the main discontinuity sets apparent in the photograph.

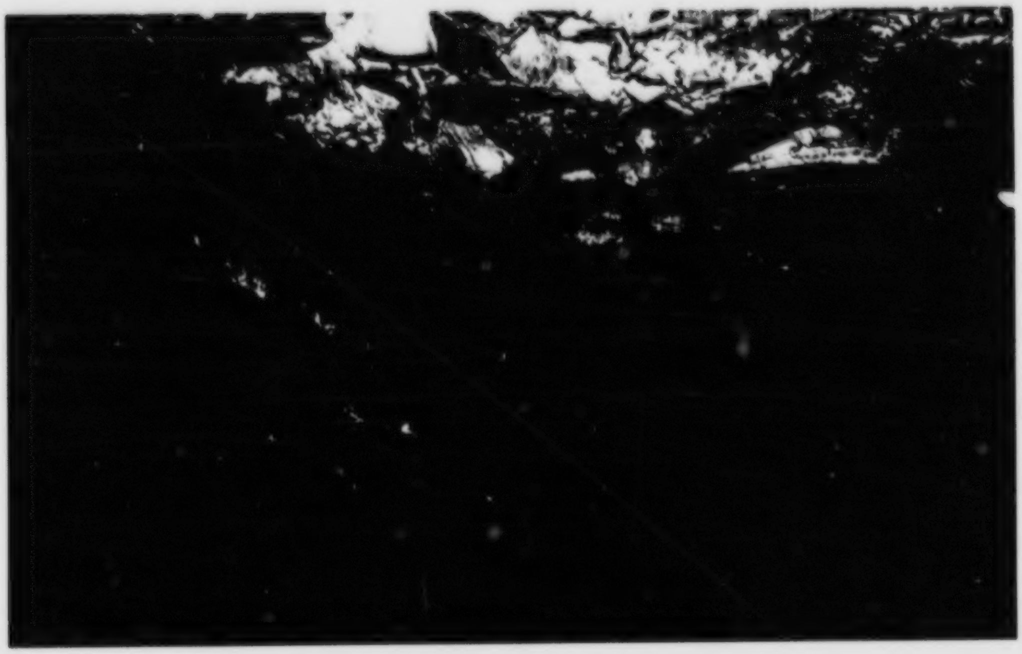


Mine 1, gallery 6: 3.1 fractures/metre. RQD = 96%
3 major sets, dip = 60°, dip = 20° to the south and dip = 20° to the north.

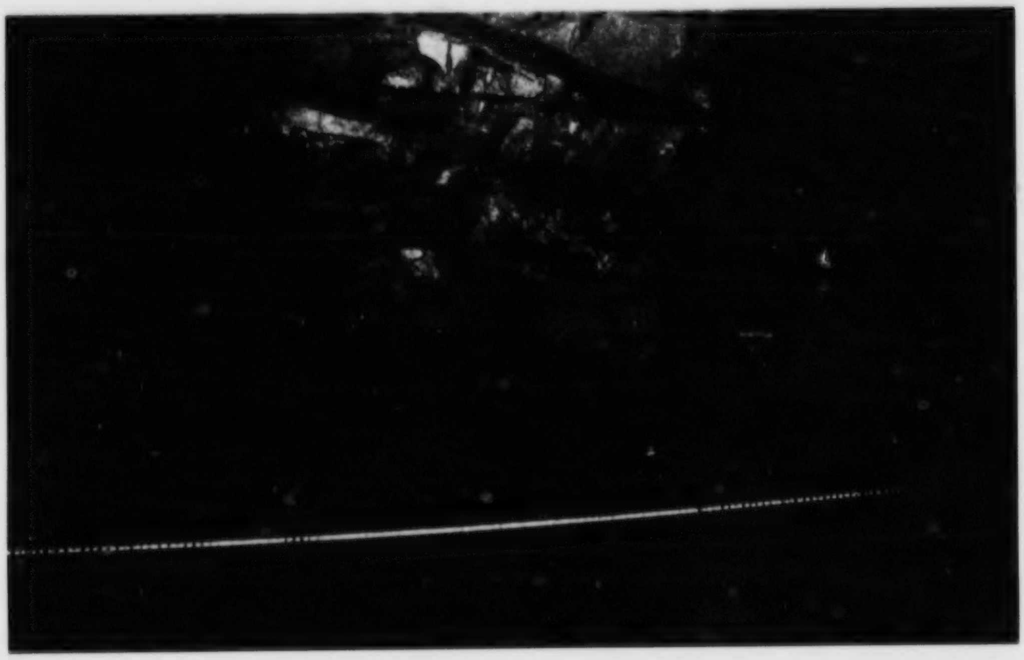
Mine 2, gallery 4: 2.0 fractures/metre. RQD = 98%
3 minor sets, dip = 80°, dip = 45°, dip = 20°

Fig. 51 – Types of ground observed during the measuring campaign carried out to study rock bolting techniques in 10 underground mines (18).

Note: Major set: over 3 m in size
Minor set: under 3 m in size

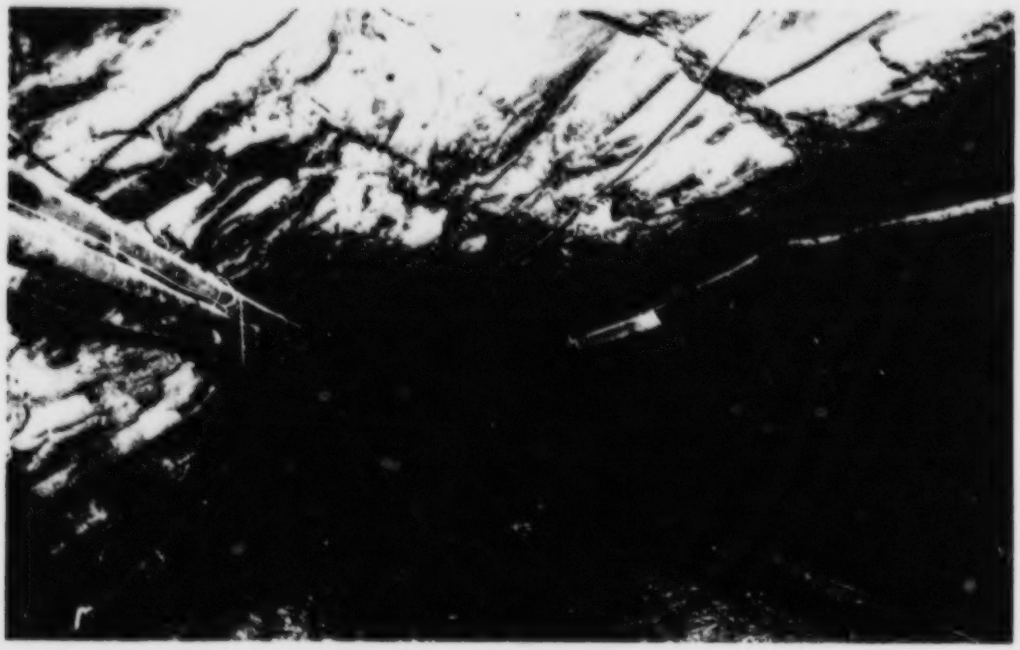


Mine 3, gallery 1: 4.9 fractures/metre, RQD = 91%
2 major sets, dip = 80° and dip = 30°
Some minor sets

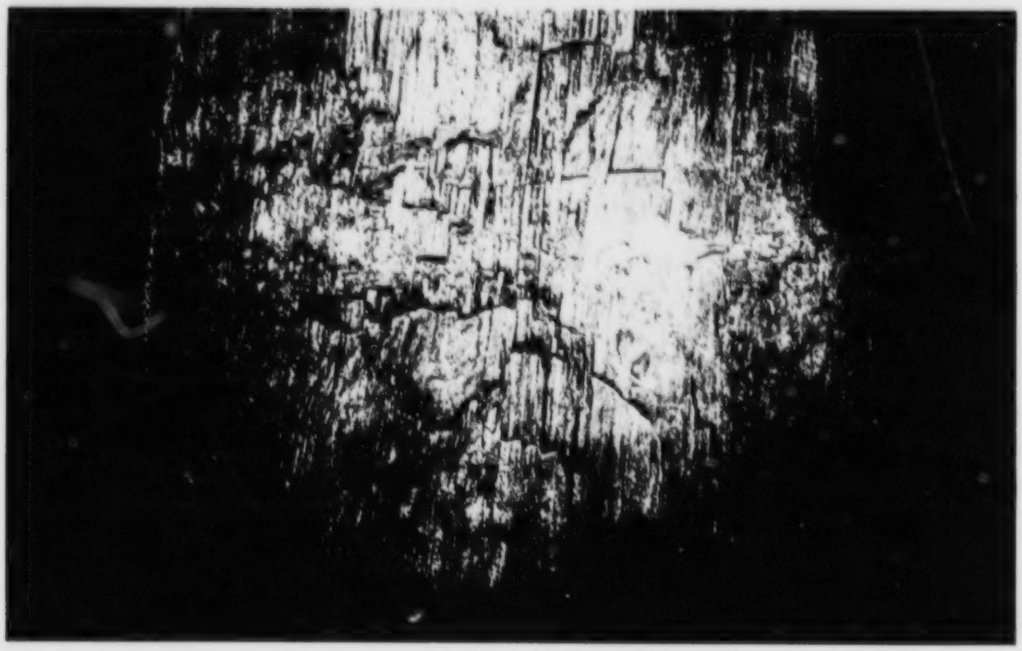


Mine 3, gallery 6: 3.0 fractures/metre, RQD = 96%
2 major sets, dip = 20° and dip = 0°

Fig. 51 – (cont'd)

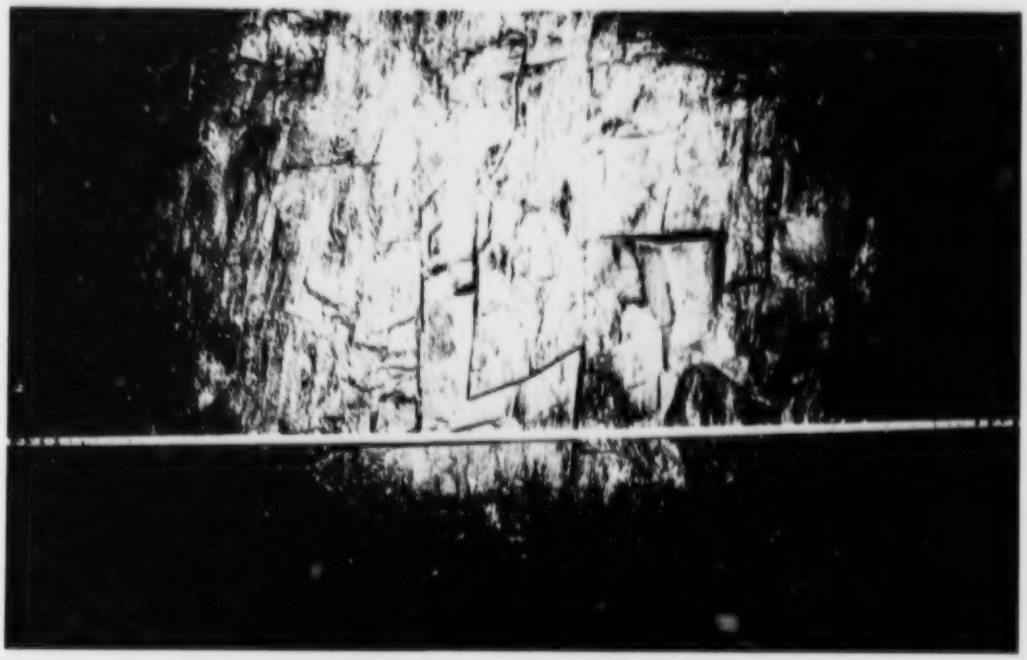


Mine 4: 2 major sets, dip = 40° and dip = 30°

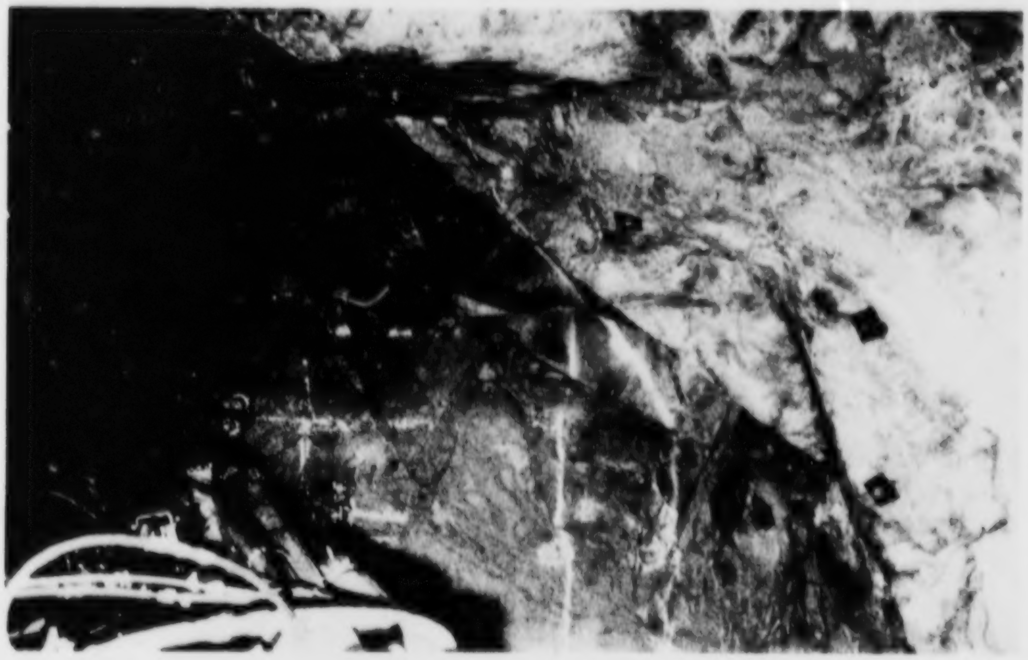


Mine 9, gallery 4: 20 fractures/metre (in this location), RQD = 40%
2 major sets, dip = 90° (schistosity), and dip = 0°

Fig. 51 – (cont'd)



Mine 9, gallery 4: 3.5 fractures/metre (in this location), RQD = 95%
3 major sets, dip = 90°, dip = 20°, dip = 0°



Mine 10: 3 major sets, dip = 50°, dip = 20° to the east, dip = 20° to the west

Fig. 51 – (cont'd)

In more complex situations where it is difficult to arrive at a classification, or in situations where we want to take into account a large number of discontinuities in a discrete area of the mine (not the local scale as previously described), a survey of discontinuities using a compass and subsequent plotting on a stereographic projection becomes necessary. The method to use in this case is described in Reference 66, and the stereographic projections can be plotted either manually (21, p. 70–71) or with the help of computer programs (67, 68).

4.2.4 Geomechanics Ground Classifications

In hard rock mines, the main geomechanics ground classifications that are presently widely used are the following:

a) the CSIR classification (South African Council for Scientific and Industrial Research), developed by Bieniawski (69);

b) the NGI classification (Norwegian Geotechnical Institute), developed by Barton, Lien, and Lunde (70);

c) the Laubscher classification (71).

The main characteristics of each of these classifications are shown on the table in Figure 52. Particular emphasis is given to the area of application and the main parameters taken into account by the authors.

It is important to point out that, of the three classifications, only one was developed specifically for mining applications, that is, the Laubscher classification. However, during the course of the last few years, it has become evident that the other two classifications have been largely used in underground mines, particularly in Canada, where the Laubscher classification is not widely employed, except in asbestos mines that are operated on the basis of block caving (72).

Classification	Year of development	Characteristics	Area of application	Main parameters							
				Uniaxial compressive strength	RQD	Joint spacing	Joint dip	No. of joint sets	Joint condition	Under-ground water inflow	Effect of weak. shear. or high stress zones
CSIR	1973	Developed on the basis of experience acquired in South Africa	Tunnels. mines. foundations	x	x	x	x		x	x	
NGI	1976	Developed on the basis of the study of 200 underground excavations and tunnels in Scandinavia	Tunnels		x			x	x	x	x
Laubscher	1977	Developed on the basis of experience acquired in the asbestos mines of Rhodesia	Mines	x	x	x		x	x	x	

Fig. 52 – Area of application and parameters of the main geomechanics classifications presently used

On the other hand, to make the CSIR classification applicable to other mining situations, Laubscher and Taylor (73) have proposed adjustmentss that can be made to the classification to take into account the following conditions: rock weathering, orientation of the stresses in relation to the main joints, stress variations as a function of mining progression, the dip of the joints and blasting effects. Even though the use of these corrections requires practice on the part of the user, it is necessary to use them in cases where we want to take into account specifically one or more of the conditions described above. It is also important to point out that Laubscher (71) uses the same adjustmentss with his own classification when he employs it to determine the amount of support required in the galleries.

The tables used to determine the three geomechanics classifications are shown in Appendix 22, along with the adjustmentss made by Laubscher and Taylor to the CSIR classification. These tables are sufficiently complete to make it possible to evaluate the classifications. Nevertheless, when using them for the first time, it is important to check one's own evaluation of the various parameters against those obtained by someone else with more experience in this area.

The three geomechanics classifications were used in Appendix 21 to evaluate the quality of the ground in the 57 gallery sections investigated in 1985. This field campaign made it possible to evaluate the performances of the three classifications and acquire experience using them in igneous rocks in the Canadian Shield. As noted in the appendix, the classifications were used alone and then, in the case of the CSIR and Laubscher classifications, with the adjustmentss introduced by Laubscher and Taylor. The main observation derived from the large-scale use of these geomechanics classifications was that there was no systematic relationship between the bolting densities used and the quality of the grounds. However, there was a proportionality trend between the two parameters. We noted also that for grounds classified with similar ratings on the basis of the geomechanics classifications, bolting density can vary in a ratio of 1 to 2.5 in two different mines.

At the same time, if we want to use the geomechanics classifications to decide whether bolting is required in a gallery, as discussed in Section 4.3, the only two geomechanics classifications that produced results consistent with field observations were the following:

- Laubscher Classification, with the Laubscher and Taylor adjustmentss;
- NGI Classification.

Thus, these two classifications will henceforth be recommended as being the most useful and widely applicable in Quebec mines and generally in mines located in the Canadian Shield. The improvement of these two classifications over the CSIR classification can be explained by a review of the table shown in Figure 52 where, contrary to the two others, the CSIR does not take into account the number of joint sets or discontinuities present, even though this parameter probably has the greatest effect on ground conditions. Moreover, the Laubscher classification makes it possible to weigh separately the joint spaces in each set (Appendix 22), which represents an additional advantage over the other classifications.

4.3 CHOICE OF BOLT TYPE

The four main types of bolts were discussed in detail in Chapter 3. Their characteristics, compatibility with various types of ground, method of installation, and performance after installation and as a function of time, were amply discussed there. The purpose of this section is not to review points already studied, but rather to discuss criteria or considerations that make it possible to choose the most suitable type of bolt. Once this choice has been made, it is still necessary to take into account the various elements discussed in Chapter 3. This will confirm the validity of the choice or provide direction for choosing another bolt that would have fewer disadvantages in the eyes of the users, either because of the installation methods used or for any other reason.

4.3.1 Choice as a Function of Ground Conditions

The main question to be decided as a function of the ground conditions is whether it is necessary to tension the bolts.

Although tensioned mechanically anchored bolts are widely used in Quebec mines, both because of their relatively low cost in comparison with other types of bolts (Section 4.3.4) and because of their generally very satisfactory performance in hard rocks, they are not the only type of support that can be used in mine galleries. An increasing number of tests on other types of fully anchored (resin- and cement-grouted bolts, Swellex and Split Set friction bolts), as well as theoretical considerations that have been confirmed in the field (Section 2.3.1), have shown that these fully anchored bolts have a different effect that makes it possible to use them also to stabilize the roof and walls of underground excavations.

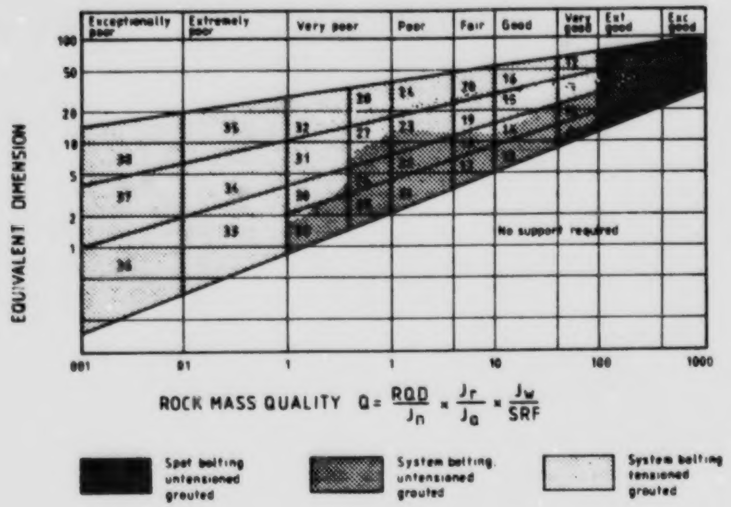
An initial conclusion from the tests discussed in Chapter 3 is that all types of bolts can be used in rocks with a compressive strength over 43 MPa (6,000 psi). By strictly following the rules stated in Chapter 3 (tolerance of the drill hole diameter for the given type of bolt, minimum grouting length for fully anchored bolts), it is possible to obtain holding powers equal to the rupture load of the bolt barrel. For rocks with a compressive strength lower than this value, pullout tests in the field are needed to find the most suitable type of bolt.

The fact that a pullout test produces satisfactory results does not mean, however, that this single criterion can be used to determine the choice of bolt. The effect of bolting, depending upon whether it is used to suspend rock blocks (Section 2.1), to solidify strata by controlling convergence (Section 2.2), or to prevent the movement of fissures in fractured ground (Section 2.3), must also be taken into account. The general considerations regarding the type of bolt recommended in each of the three cases are discussed in the sections indicated.

For preventing the movement of fissures in fractured grounds, the experience of Quebec mines discussed in Appendix 21 seems to confirm that the two types of bolt (tensioned mechanically anchored bolts and non-tensioned fully anchored bolts) are used relatively indiscriminately in the generally average-quality grounds under consideration. Nevertheless, the fully anchored bolts are used mostly in lower quality grounds, as well as in locations used for specific purposes (draw points, crushing chambers, etc.).

The type of bolt can also be chosen based on the geomechanics ground classifications. The authors of the various classifications include recommendations regarding the type of bolt to use and the required bolting density (Section 4.4.2). However, these authors generally prefer fully anchored bolts, in accordance with the mechanism of creating a natural rock vault that was discussed in Section 2.3.1. Thus, Laubscher (71) recommends the use of fully anchored bolts regardless of the conditions of the ground, except for those of very poor quality (a rating of less than 40, after adjustments using the Laubscher and Taylor adjustments). In poor-quality grounds he recommends the use of shotcrete or a lining of concrete poured in place, in addition to the rock bolts (Section 4.4.2).

As far as the authors of the NGI classification are concerned, the table in Figure 53 can be used to decide whether to use tensioned bolts (22, 70). It appears that non-tensioned fully anchored bolts are preferred in all excavations with an equivalent dimension under 10 m and a Q rating higher than 1. For excavations with a Q rating between 0.1 and 1, the minimum equivalent dimension below which tensioned bolts are recommended varies from 2 to 10 m. It is important to point out that, in the opinion of the authors, tensioned bolts should also be grouted with cement, since the NGI classification was mainly developed for tunnels and other permanent underground installations. Given the widths under consideration (over 5 m), we consider that the NGI recommendations could be especially useful when choosing a support using cable bolts, whether tensioned or not, for stopes rather than for narrow mining galleries.



$$\text{EQUIVALENT DIMENSION} = \frac{\text{span of the opening}}{\text{ESR}}$$

with ESR = 3 to 5, temporary mining openings
ESR = 1.6, permanent mining openings
ESR = 1.0, crushing chamber

Fig. 53 – Type of bolt recommended (tensioned or non-tensioned) as a function of the NGI geomechanics ground classification. From Barton *et al.* (70).

4.3.2 Choice Based on the Observation of the Performance of Bolts in the Field

One of the main disadvantages often mentioned regarding the use of mechanically anchored bolts is the crumbling of the rock under the head plate, which leads to a loss of tension in the bolt.

This effect was very often observed during visits to 13 Quebec mines in 1984 (16), several of which reported a percentage of unserviceable bolts of more than 10%. This finding illustrates that it is necessary to choose a type of bolt that is compatible with the ground. When mechanically anchored bolts do not provide a satisfactory solution, an initial corrective step could be to change the head plate to a larger plate (127 x 127 mm or 152 x 152 mm; 5" x 5" or 6" x 6") such as discussed in Section 3.1.3. Nevertheless, this measure could prove insufficient to reduce significantly the percentage of unfit bolts, and it may then become necessary to use fully anchored bolts.

Similarly, fully anchored bolts could suffer high percentage losses. The best example is that of mine 9 visited in 1984 (16), which reported the changes discussed below in the types of support used since the opening of the mine.

This mine is operated by the retreating sublevel method. The deposit is present in a tabular form and is contained in a sequence of almost vertical volcanic formations. The sublevel galleries are opened in the mineralized zone and are parallel to the seam, while the development galleries are perpendicular to them.

All the sublevel galleries as well as some development galleries near the deposit are under strong ground pressures because of the exploitation method used. In many cases, the walls converge significantly. This is caused by the buckling of layers that are very schistous.

At the time the mine started to operate, and the ground was found to be of poor quality, resin-grouted bolts of various lengths (1.5 m, 2.5 m, 3 m; 5 ft, 8 ft, 10 ft) were used in the sublevel galleries. However, rupture of these bolts reached 20%, because of the convergence of the walls. The resin-grouted bolts were too stiff and did not stretch sufficiently to accommodate the convergence. Thus, they were replaced by 2.5 m (8 ft) and 3 m (10 ft) Swellex bolts which proved to be more satisfactory, with less than 1% rupture losses. In this case, the Swellex bolts seem to be more easily deformable and compatible with the ground, even though their holding power is probably less than that of resin-grouted bolts.

In development galleries located in waste areas, the initial support consisted of mechanically anchored bolts and Split Set bolts 1.5 m (5 ft) long. Problems were encountered with these two types of bolts in relation to wall movements. Among other things, the heads of Split Set bolts could yield suddenly. The mine has now opted for the Split Set bolts in all haulage galleries for standardization.

Thus, the cause of rupture of bolts in the mine was the excessive convergence of the walls. The sudden rupture of bolts of all types can also be caused by minor ground bursts commonly called "bumps." These are shocks of relatively low intensity caused by small rock displacements. In several of the mines visited in 1984 (16), particularly in mine 12, there were occasional rock falls from the roof or walls of the galleries, and the bolts had been cut by these shocks. Thus, a true rock burst will be even more likely to cut the bolts and will send rock flying into the gallery.

For a given mine, it is possible that the use of more rigid support obtained by increasing the diameter of the mechanically anchored bolts or resin-grouted bolts would make it possible to reduce the incidence of broken bolts. However, the experience acquired in locations where rock bursts are likely to occur shows that the mine will eventually have to opt for a more yielding type of support, more compatible with the amount of energy released by the ground that must be absorbed by the support.

These various situations - rupture and loss of bolts caused by the crumbling of the rock under the support plate, by excessive convergence of the ground, and by the incidence of rock bursts - show that the parameter of the deformability of the support bolts is as important as their holding power when we want to choose the most suitable type of bolt for a particular location. For this reason, a scale showing the stiffness of types of bolts, as well as a proposed utilization method, are discussed in the section below.

4.3.3 Stiffness Scale of Various Types of Bolts

The stiffness of a bolt is normally determined by a pullout test that is carried out in the field and is identical to those described in Section 3.1.9. It is calculated by the slope of the pullout curve drawn between the origin and the point of rupture.

It is important to determine the stiffness of the bolt using a field test. Laboratory tests, where the bolt is attached to a load frame in traction, do not include the parameters of anchor slip in the drill hole, or interlocking of the barrel in the case of a grouted bolt.

The table in Figure 54 classifies the types of bolt by increasing order of stiffness. The stiffness values proposed are only approximate, since they were obtained from several sources identified in the table. The real values for a particular site could vary because of the nature of the ground, the testing method used, or whether the bolts tested have been properly installed. For this reason, it is very important to carry out these tests on the site itself to determine the true stiffness values that can be safely expected.

The following method for using the stiffness table is recommended to ensure that the bolts used are compatible with the type of ground where they are installed.

a) Carry out systematic gallery surveys of the percentage of bolts that have become unfit because of rock crumbling under the head plate (case A),or of bolts that have been broken by excess tension caused by convergence of the ground (case B), or of bolts that have been sheared through the effect of rock bursts (case C).

b) For case A, the acceptable Figure should not be more than 5%. Beyond this value, it is necessary to change the head plates (Sections 3.1.3, and 4.3.2) or plan to use fully anchored bolting.

c) For cases B and C, the acceptable Figure should be below 1% because of the violence with which ruptures can occur. If the percentage obtained is above this value, one could plan to reduce it by using bolts that are not as stiff (or a combination of two different types of bolts). These can be chosen by moving up the table. In many cases, before changing the type of bolt, it may be possible to modify the diameter of the drill hole where they are installed, since this can considerably change the stiffness values as they appear on the table.

Type	Stiffness (kN/m)	Source
Split Set bolt	2,000 to 3,000	Reference A.19
Mechanically anchored bolt Diameter: 15.9 mm (5/8")	3,500 to 5,250	Appendix 11, Reference A.12
Mechanically anchored bolt Diameter: 19 mm (3/4")	5,250 to 8,500	Estimate, Reference 38
Resin-grouted bolt Diameter: 19 mm (3/4"), grouted in a 31.75 mm (1 1/4") hole	7,800	Appendix 11, Reference A.14
Swellex bolt in a 31.75 mm (1 1/4") hole	8,500 to 12,000	Estimate, References 52, 58
Swellex bolt in a 38.1 mm (1 1/2") hole	22,500 to 26,000	Estimate, References 52, 58
Cement-grouted bolt Diameter: 19 mm (3/4"), grouted in a 31.75 mm (1 1/4") hole	26,000 to 35,000	Estimate, Reference 58
Resin-grouted bolt Diameter: 19 mm (3/4"), grouted in a 25.4 mm (1") hole	70,000	Appendix 14, Reference A.14

Fig. 54 – Stiffness scale for the main types of bolts

4.3.4 Comparing the Relative Costs of the Main Types of Bolts

When many types of bolts can be used to obtain adequate support for a given type of ground, the final choice could be dictated by cost. The approximate relative costs (1986) of 2.1 m (7 ft) long bolts of each type equipped with their accessories are shown in Figure 55. These costs do not include the installation of the bolts which, in general, is about the same from one bolt to another. The installation cost could amount to a relative cost of about 300 that should be added to the costs shown in the Figure.

Type of Bolt	Relative cost
Mechanically anchored bolt with head plate	100
Resin-grouted bolt with head plate	232
Cement-grouted bolt with head plate	128
Cement cartridge grouted bolt with head plate	214
Swellex friction bolt with head plate	237
Split Set friction bolt with head plate	220

Fig. 55 – Relative cost of the main types of bolts

4.4 CHOOSING THE SPACING AND LENGTH OF BOLTS

Methods that can be used to choose the installation patterns and bolt lengths will be reviewed later. Nevertheless, an initial verification is described in the section below to confirm whether rock bolting is necessary.

4.4.1 Verification of the Need for Rock Bolting

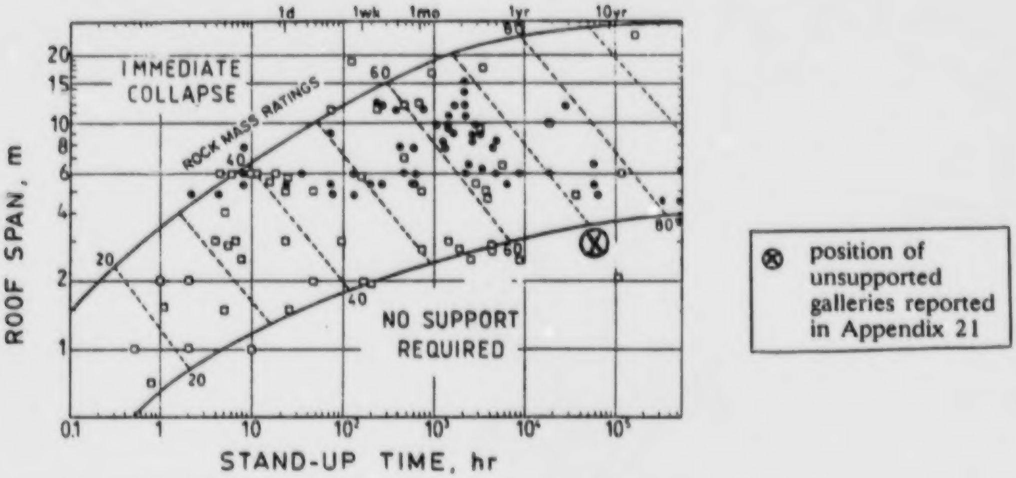
The need for rock bolts in a gallery or stope may be evaluated using the geomechanics classifications. Two graphs, one produced by the authors of the NGI classification (70), and the other by the author of the CSIR classification (14, 69) are very widely used. These graphs were prepared by a back analysis of various mining and civil engineering installations that have shown various degrees of stability. These two graphs are shown in Figure 56.

The CSIR graph was developed based on 56 cases of roof collapse in mining installations, and 54 case histories in civil engineering installations that were studied to estimate the expected lifespan of the excavation (stand-up time) without rock falls. These graphs produce estimates that are clearly more pessimistic than those obtained with the NGI graph, developed on the basis of 30 case studies of civil engineering excavations that were found to be stable over the years.

Thus, in accordance with the CSIR graph, it would not be possible to open a gallery more than 4 m (13 ft) wide without support, even in grounds with a very high rock mass rating (RMR), 80 or more. This RMR = 80 rating is equivalent to a Q = 55 rating using the NGI equivalence formula shown below (14, p. 128).

$$\text{RMR} = 9 \ln Q + 44$$

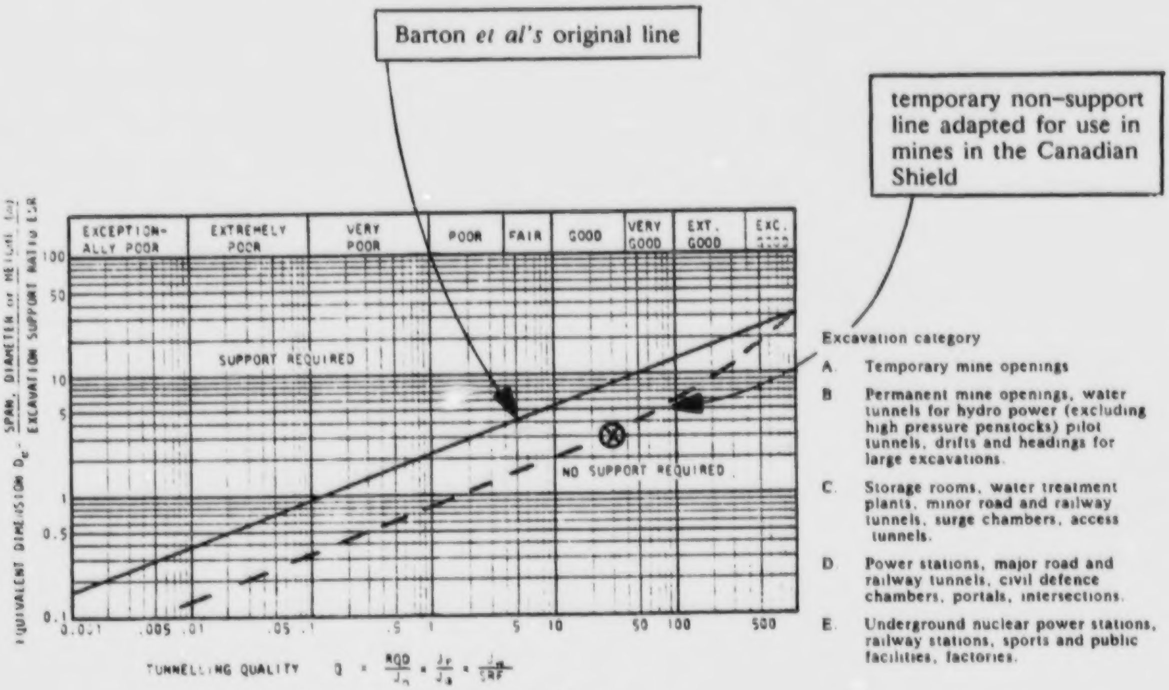
In the case of grounds with a Q = 55 rating, the maximum gallery width at which it is necessary to start supporting the opening can be read on the graph on Figure 56, taking into account that ESR = 1.6. Thus, this maximum width would be 16 m (52.5 ft), according to Barton *et al.*'s original non-support line.



⊗ position of unsupported galleries reported in Appendix 21

Geomechanics Classification of rock masses output for mining and tunneling. ● = case histories of roof fails in mining. □ = tunneling roof falls. contour lines = limits of applicability

a) Estimate using the CSIR classification



Barton *et al's* original line

temporary non-support line adapted for use in mines in the Canadian Shield

Excavation category		ESR
A.	Temporary mine openings	3 - 5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks) pilot tunnels, drifts and headings for large excavations.	1.6
C.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D.	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections.	1.0
E.	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

b) Estimate using the NGI classification

Fig. 56 – Estimates of the need for support in galleries and stopes using the CSIR and NGI geomechanics classifications

To discriminate between the two classifications in terms of the accuracy of their forecasts regarding the need for support in Quebec mines, we found it useful to verify them against the observations made in the 57 gallery sections visited in 1985 (Appendix 21). During this campaign, seven unsupported galleries (generally 3 m (10 ft) wide) were investigated and their CSIR and NGI classifications established. These classifications were equal to or higher than RMR = 70 and Q = 30 respectively (see Figure A.43 in Appendix 21), and the corresponding points are plotted on the two graphs on Figure 56 where they are indicated by ⊗. In both cases, they are in areas where it is estimated by the authors of the CSIR and NGI classifications that support is not necessary. With these observations, we could draw the following conclusions on the use of these two graphs:

- Although appearing pessimistic, the CSIR graph is still in accordance with the trends observed in Quebec mines, since the point plotted is very close to the no-support-required limit established by it. Nevertheless, as we point out in Appendix 21, the Laubscher classification should be used in preference to that of the CSIR, even to evaluate the need for support with the help of the CSIR graph, because it is more suitable for use in the geological context of mines in the Canadian Shield.

- The use of the NGI graph in its present form cannot be recommended, because it will open the way to situations that are very different from current practices in Quebec mines. However, as we point out in Appendix 21, the NGI classification is very compatible with the geological context of mines in the Canadian Shield. Thus, the NGI graph should be revised based on a large number of case studies peculiar to this context. Initially, a recommendation that could be followed to make the graph more useful is to lower the authors' original line indicating that no support is required, so that the line passes through the point corresponding to the seven unsupported mine galleries studied in Quebec. This new no-support-required line, which is temporary until a larger number of case studies have been conducted, is drawn using a broken line on the graph in Figure 56.

Finally, it is useful to point out that the ground scaling operation has lost none of its importance, particularly in galleries where, on the basis of geomechanics classifications, it is possible to recommend that no support is necessary. The safety principles related to this operation (74) are reviewed in Figure 57.

1. Use scaling bars of suitable length as needed.
2. Ensure that the environment is not too noisy during the scaling operation.
3. Stand on a solid surface that is as regular as possible and has a large clearance around it.
4. Make sure to leave a free space behind you so that you can back off quickly.
5. Ensure that the rock dislodged falls in such a way that it does not hit you, your companions, or the equipment.
6. Proceed with the scaling operation from sound to unsound ground.
7. Remember that a rock may fall as soon as it comes in contact with the bar.
8. If motorized equipment is used as a working platform, it should be equipped with adequate devices to ensure staff safety.
9. Ensure safe working conditions in your work station.

Fig. 57 – Safety principles for ground scaling (73)

4.4.2 Choosing the Spacing and Length of Bolts on the Basis of Geomechanics Classifications

The most useful geomechanics classifications to estimate the spacing and length of the bolts are the NGI and CSIR (or Laubscher's) classifications, the latter used with the Laubscher and Taylor adjustmentss.

4.4.2.1 NGI Classification

Barton *et al.* (70) have proposed the formulas shown in Figure 58 to estimate the support pressures P_{roof} and P_{wall} required in the roof and walls of underground excavations, as well as the length of the bolts L_{roof} and L_{wall} that should be used. The following observations may be made:

- After careful examination of measurements of load cells installed between supports and the rock face in more than 26 large caverns, Barton *et al's* conclusion is that the roof and wall support pressures P_{roof} and P_{wall} are not dependent on either height or span of the excavation. Consequently, these latter parameters do not appear in Figure 58 for computation of support pressures, and the bolt spacing obtained from the formula presented below will be the same for a large or a small excavation.

- The spacing of the bolts can be calculated with the help of the support pressure P_{roof} or P_{wall} using the following formula:

$$S = \sqrt{\frac{C.10^{-3}}{P_{roof} \text{ or } P_{wall}}}$$

where C: load exceeding the yield strength of the bolt (see Sections 3.1.1, 3.2.2, 3.3.1, and 3.4.1), in kN

S: spacing of the bolts, in metres

- The above formula contains a safety factor, because it recommends the use of the load exceeding the yield strength of the bolts, and not their load at rupture.

- The length of bolts L_{roof} and L_{wall} in Figure 58 is suggested according to ealier work by Benson *et al* (75).

Support pressure	Length of the bolts
$P_{roof} = \frac{0.2}{Jr} \cdot Q^{-1/3}$, if the number of discontinuity sets > 2	$L_{roof} = 2 + 0.15\ B/ESR$
$P_{roof} = \frac{0.2\ Jn^{1/2} \cdot Q^{-1/3}}{3.Jr}$, if the number of discontinuity sets ≤ 2	
P_{wall} calculated with the same formulas as P_{roof}, by replacing Q by Q', with: $Q' = 5\ Q$ if $Q > 10$ $Q' = 2.5\ Q$ if $0.1 \leq Q \leq 10$ $Q' = Q$ if $Q < 0.1$	$L_{wall} = 2 + 0.15\ B/ESR$
Notes: Jr (joint roughness number) determined as in Appendix 22; Jn (joint set number) determined as in Appendix 22; Q, NGI classification rating, determined as in Appendix 22; P, support pressure, in MPa; B, span of the excavation (m); ESR (excavation support ratio), see Figure 56, generally equal to 1.6 for permanent mining excavations; L, length of the bolts (m).	

Fig. 58 – Support pressure and length of the bolts determined on the basis of the NGI geomechanics classification (70)

- When the calculated P_{wall} is less than 0.04 MPa, which corresponds to the support pressure of mechanically anchored bolts with a diameter of 15.9 mm (5/8") installed in a pattern of 1.2 m x 1.2 m, bolting the walls ceases to be effective and can be abandoned.

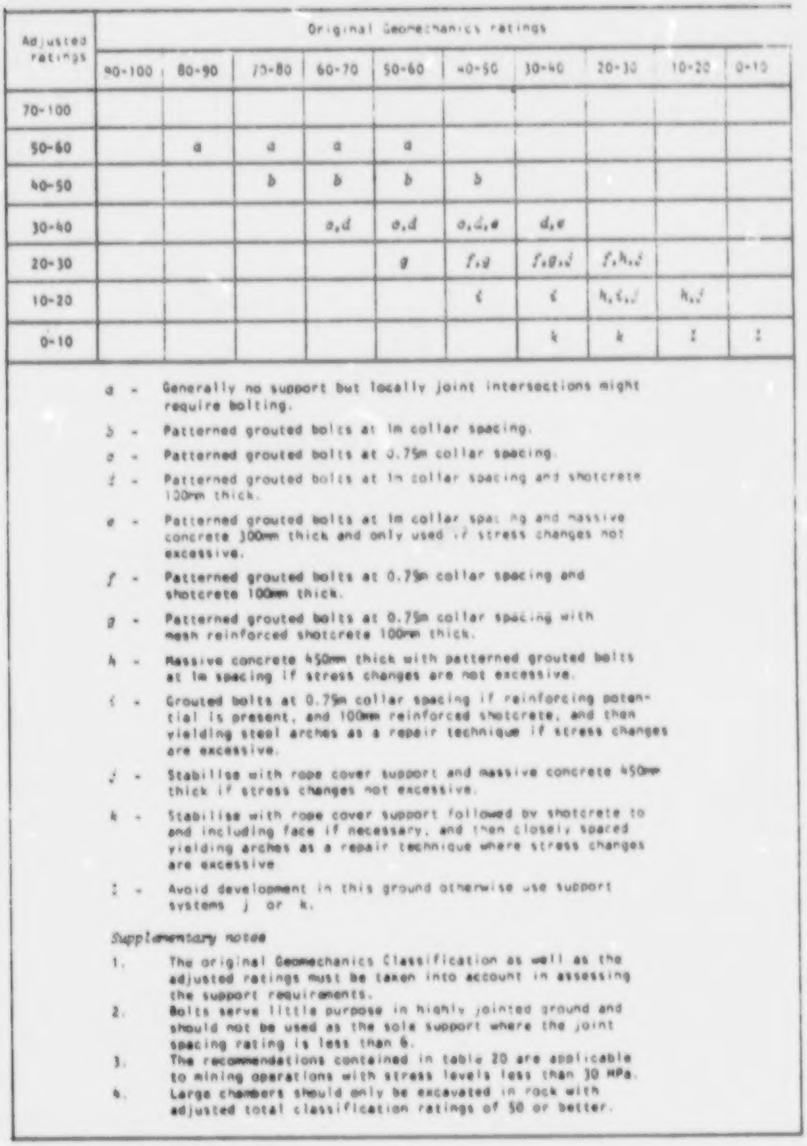
- When Q is lower than 10, it may happen that the calculated P_{roof} becomes so high that the resulting low rockbolt spacing becomes uneconomical. In such instances, cable bolts which have a higher yield strength or shotcrete used according to recommendations of Barton *et al.* (70) (21, p. 290 to 295) should be preferred.

4.4.2.2 Laubscher Classification

At the same time as they introduced adjustmentss to the Laubscher and CSIR classifications, Laubscher and Taylor (73) produced a support selection grid that is reproduced in Figure 59. This grid is used by evaluating the initial CSIR rating (or preferably the Laubscher rating as discussed in Section 4.2.4) and then evaluating the rating with the adjustmentss introduced by Laubscher and Taylor. The intersection of the two ratings on the grid gives the recommended support.

4.4.3 Minimum Rock Bolting Density Recommended for Mines in the Canadian Shield

As part of the measurement campaign in ten Quebec mines, rockbolting densities, e.g., the number of bolts per square meter of roof and wall if bolted, were evaluated in more than 57 drift portions (17, 18). At the same time, the various parameters needed for evaluating the NGI and the Laubscher (with the Laubscher and Taylor adjustments) rock mass classification were also recorded at the exact same locations in the drifts. The resulting comparison of the two parameters is shown in Figure 60, for the NGI classification only (see Appendix 21 for comparison with Laubscher classification).



Adjusted ratings	Original Geomechanics ratings									
	90-100	80-90	70-80	60-70	50-60	40-50	30-40	20-30	10-20	0-10
70-100										
50-60		*a*	*a*	*a*	*a*					
40-50			*b*	*b*	*b*	*b*				
30-40				*c,d*	*c,d*	*c,d,e*	*d,e*			
20-30					*g*	*f,g*	*f,g,j*	*f,h,j*		
10-20						*i*	*i*	*h,i,j*	*h,j*	
0-10							*k*	*k*	*l*	*l*

a - Generally no support but locally joint intersections might require bolting.

b - Patterned grouted bolts at 1m collar spacing.

c - Patterned grouted bolts at 0.75m collar spacing.

d - Patterned grouted bolts at 1m collar spacing and shotcrete 100mm thick.

e - Patterned grouted bolts at 1m collar spacing and massive concrete 300mm thick and only used if stress changes not excessive.

f - Patterned grouted bolts at 0.75m collar spacing and shotcrete 100mm thick.

g - Patterned grouted bolts at 0.75m collar spacing with mesh reinforced shotcrete 100mm thick.

h - Massive concrete 450mm thick with patterned grouted bolts at 1m spacing if stress changes are not excessive.

i - Grouted bolts at 0.75m collar spacing if reinforcing potential is present, and 100mm reinforced shotcrete, and then yielding steel arches as a repair technique if stress changes are excessive.

j - Stabilise with rope cover support and massive concrete 450mm thick if stress changes not excessive.

k - Stabilise with rope cover support followed by shotcrete to and including face if necessary, and then closely spaced yielding arches as a repair technique where stress changes are excessive

l - Avoid development in this ground otherwise use support systems j or k.

Supplementary notes

1. The original Geomechanics Classification as well as the adjusted ratings must be taken into account in assessing the support requirements.
2. Bolts serve little purpose in highly jointed ground and should not be used as the sole support where the joint spacing rating is less than 6.
3. The recommendations contained in table 20 are applicable to mining operations with stress levels less than 30 MPa.
4. Large chambers should only be excavated in rock with adjusted total classification ratings of 50 or better.

Fig. 59 - Support selection grid. From Laubscher and Taylor (73)

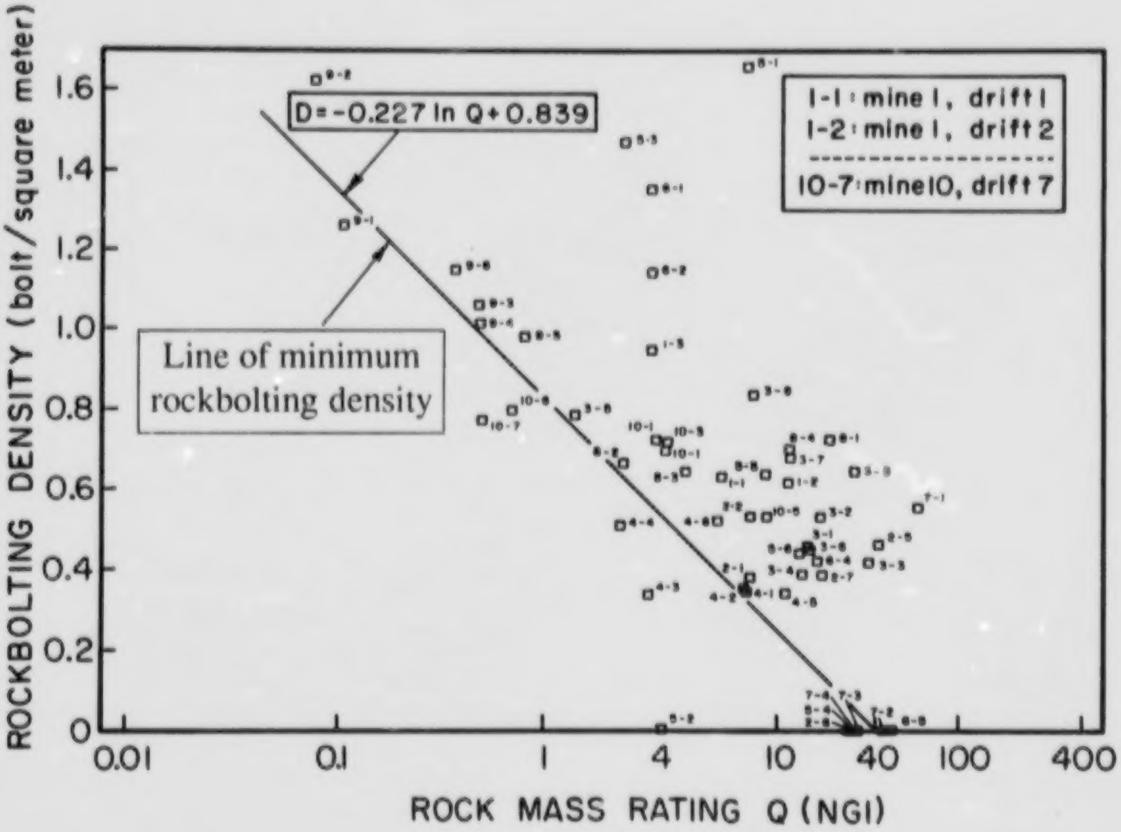


Fig. 60 – Line of minimum bolting density observed in mines of the Canadian Shield

It was then thought of interest to superimpose on the graph a line that would be located near the base of all points and that would then represent the minimum bolting density required in mining situations (see Appendix 21 for more details). This line can be considered as representing the minimum amount of support judged as necessary, according to the experience of the ten mines visited. It is obvious that there might also be some situations of overdesign in the drifts surveyed. This is characterized by the fact that many points are well above the line. The equation of the line of minimum required support as follows:

$$D = -0.227 \ln Q + 0.839$$

where:

D: number of bolts per square meter of roof and wall, if the latter is bolted

Q: rating of rock mass according to NGI classification

The following remarks can be made regarding the use of the above equation.

- Span of drifts surveyed varied between 2.8 m and 7.5 m, with a majority between 3.5 and 5.5 m. However, if the assumption of Barton *et al.* is correct, according to which the support pressure needed is not dependent on excavation span, the equation can also be used for support assessment of larger excavations. Depth of drifts surveyed varied between 50 m and 1000 m, with a majority between 100 m and 500 m.
- Value of D can be converted to bolt spacing by the following formula.

$$S = \frac{1}{D^{1/2}}$$

where:

S: bolt spacing, (m)

- From Figure 60, it can be seen that an unsupported excavation is feasible in a rock mass with a rating Q above 40, although some spot bolting could be necessary, depending on local situations. A more detailed description of the relationship between the NGI classification and the possibility of leaving a gallery unsupported is already given in Section 4.4.1.

4.4.4 Choosing Spacing and Length of Bolts on the Basis of Empirical Rules

Two sets of empirical rules reported in recent litterature for the determination of length and spacing of rockbolts will be discussed: that of the U.S. Corps of Engineers (7) and that of Farmer and Shelton (76). Another set of rules was developed by Lang (77) in 1961, which then gained wide acceptance and was reproduced in many rock mechanics textbooks. However, the authors of the two more recent sets of rules partially based their work on Lang's, consequently they do not need to be reproduced here.

4.4.4.1 Rules of the U.S. Corps of Engineers

The rules were established by the U.S. Corps of Engineers (7) after careful examination of more than 68 case histories of rock reinforcement in underground chambers, tunnels and shafts. Width of openings surveyed varied between 4.5 m and 30 m and heights between 4 m and 60 m. Depths were moderate, generally not exceeding 150 m. No mine openings were surveyed.

These rules are presented in Figures 61 and 62. They allow for the estimation of length, spacing and support pressure. The U.S. Corps of Engineers warns, however, that the rules give a preliminary configuration for rock reinforcement, which must be checked, analyzed and, as necessary, modified to meet the requirements of a specific rock reinforcement design.

The use of Figures 61 and 62 requires, to start with, one assumed value for one of the variables, length or spacing. The next step is to verify that all specifications are met by going through the table as many times as necessary. The process is actually short and will be illustrated in a worked example in a further section.

Figure 62 provides values of support pressure which can be directly used for calculation of bolt spacings of Section 4.4.2.1. The working load of bolts to be used should be at yield point, as assumed by the Corps of Engineers. Taking a fraction of the yield limit load of the bolts would bring an additional factor of safety to the one already included in the projects surveyed.

4.4.4.2 Rules of Farmer and Shelton

These rules were based on various author's experience, especially Rabciewicz (78), Lang (77) and Alexander *et al.* (79). They provide design guidelines for length and spacing of rockbolts for excavations in rock masses having clean, tight discontinuity interfaces and a maximum of three discontinuity sets. The rules are presented in Figure 63.

PARAMETER	EMPIRICAL RULES
Minimum length	**Greatest of:** A. Two times the bolt spacing B. Three times the width of critical and potentially unstable rock blocks* C. For elements above the springline: 1. Spans less than 6 m - 1/2 span 2. Spans from 18 m to 30 m - 1/4 span 3. Spans 6 m to 18 m - interpolate between 3 m and 4.5 m lengths, respectively D. For elements below the springline: 1. For openings less than 18 m high - use lengths as determined in C above 2. For openings greater than 18 m high - 1/5 the height
Maximum spacing	**Least of:** A. 1/2 the bolt length B. 1-1/2 the width of critical and potentially unstable rock blocks C. 1.8 m**
Minimun spacing	0.9 to 1.2 m

* Where the joint spacing is close and the span is relatively large, the superposition of two bolting patterns may be appropriate; e.g., long heavy bolts on wide centers to support the span and shorter and thinner bolts on closer centers to stabilize the surface against ravelling due to close jointing.

** Greater spacing than 1.8 m would make attachment of surface treatment such as chain link fabric difficult.

Fig. 61 - Minimum length and maximum spacing for rock reinforcement. After U.S. Corps of Engineers (7)

PARAMETER	**EMPIRICAL RULES**
Minimum average support pressure at yield point of elements	**Greatest of:** Above springline: A. Pressure equal to a vertical rock load of 0.2 times the opening width* B. 0.04 MPa** Below Springline: A. pressure equal to a vertical rock load of 0.1 times the opening height*** B. 0.04 MPa**** At intersections: Two times the support pressure as determined above

* For example, if the unit weight of the rock is 0.023 MN/m^3 and the opening span is 25 m, the required support pressure is $0.2 \times 25 \times 0.023 = 0.115$ MPa

** For the maximum spacing of 1.8 m, this requires a yield strength of approximately 142 kN

*** For example if the unit weight of the rock is 0.026 MN/m^3 and the cavity height is 45 m, the required support pressure is $0.1 \times 45 \times 0.026 = 0.117$ MPa

**** This reinforcement should be installed from the first opening excavated prior to forming the intersection. Stress concentrations are generally higher at intersections, and rock blocks are free to move toward both openings.

Fig. 62 - Minimum average support pressure for rock reinforcement. After U.S. Corps of Engineers (7)

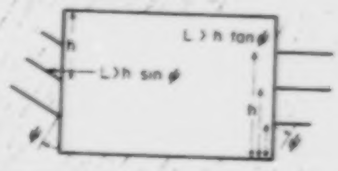
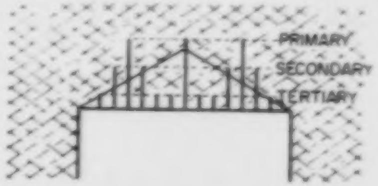
EXCAVATION SPAN (m)	NUMBER OF DISCONTINUITY SETS	BOLT DESIGN	COMMENTS
<15	≤2 inclined at 0–45° to horizontal	L = 0.3 B S = 0.5L (depending on thickness and strength of stata). Install bolts perpendicular to lamination where possible with wire mesh to prevent flaking	The purpose of bolting is to create a load-carrying beam over span. Fully bonded bolts create greater discontinuity shear stiffness. Tensioned bolts should be used in weak rock; subhorizontal tensioned bolts where vertical discontinuities occur
	≤2 inclined at 45–90° to horizontal	For side bolts: L > h sin ψ (if installed perpendicular to discontinuity; L > h tan ψ (if installed horizontally). See figure below for h and ψ; L = bolt length; s = bolt spacing; B = excavation span 	Roof bolting as above. Side bolts designed to prevent sliding along planar discontinuities. Spacing should be such that anchorage capacity is greater than sliding or toppling weight. Bolts should be tensioned sufficiently to prevent sliding
	≥3 with clean tight interfaces	L = 2s S = 3–4 x block dimension. Install bolts perpendicular to excavation periphery with wire mesh to prevent flaking	Bolts should be installed quickly after excavation to prevent loosening and retain tangential stresses. Prestresses should be applied to create zone of radial confinement. Sidewall bolting where toe of wedge daylights in side wall
>15	≤2	$L_1 = 0.3\ B_1$ primary bolting $S_1 = 0.5\ L_1$ $L_2 = 0.3\ S_1$ secondary bolting $S_2 = 0.5\ L_2$ Install wire mesh to prevent spalling	Primary bolting conforms to smaller excavation design. Secondary (and tertiary) bolting supplements primary design (See figure below)
	≥3 with clean tight interfaces	$L_1 = 0.3\ B_1$ primary bolting $S_1 = 0.5\ L_1$ $S_2 = 3/4$ x block size; secondary bolting $L_2 = 2\ S_2$	Primary bolting should have sufficient capacity to restrain major blocks. Decisions on block size for secondary bolting should be left to the section engineer 

Fig. 63 – Rockbolt parameter design rules for rock masses with ≤2 and ≤3 discontinuity sets with clean, tight interfaces. After Farmer and Shelton (76).

4.4.5 Worked Examples of Support Design

The various methods and formulae for rockbolt length and spacing evaluation will be applied to two different situations: a gallery with 3.5 m (11.5 ft) in height and 5 m (16.4 ft) in width and a chamber with 20 m (65.5 ft) in height and 15 m (49 ft) in width.

Three different qualities of rock mass will also be considered for each of the two situations, e.g., weak, average and hard rock. Their properties are given in Figure 64, in terms of the six parameters of the NGI classification system.

NGI parameter	ROCK TYPE		
	Weak	Average	Strong
RQD	30	60	95
Jn	9[1]	6[2]	3[3]
Jr	1	2	3
Ja	3	2	1
Jw	1	1	1
SRF	1	1	1
$Q = \frac{RQD}{Jn} \cdot \frac{Jr}{Ja} \cdot \frac{Jw}{SRF}$	1.11	10	95

1. Three joint sets
2. Two joint sets plus random
3. One joint set plus random.

Fig. 64 – Properties of the weak, average and strong rock types for rockbolt design

Finally, it will also be assumed that rockbolts to be used in the openings are standard 15.9 mm (5/8") diameter bolts with a yield load of 60 kN for the drift and 19 mm (3/4") diameter bolts with a yield load of 90 kN for the chamber.

The four different methods of selecting length and spacing presented in the previous sections will be illustrated below for the 3.5 x 5 m drift in the average quality rock type.

a) NGI's method (Section 4.4.2.1)

- Roof support

$$P_{roof} = \frac{0.2 \times 6^{1/2} \times (10)^{-1/3}}{3 \times 2} = 0.038\ MPa$$

$$S = \left(\frac{60.10^{-3}}{0.038}\right)^{1/2} = 1.25 \text{ m (bolt length)}$$

$$L_{roof} = 2 + 0.15 \times 5/1.6 = 2.45 \text{ m (bolt length)}$$

- Wall support

$$Q' = 2.5 \times 10 = 25$$

$$P_{wall} = \frac{0.2 \times 6^{1/2} \times (25)^{-1/3}}{3 \times 2} = 0.028 \text{ MPa} < 0.04 \text{ MPa}$$

(then, no wall support)

b) U.S. Corps of Engineer's method (Section 4.4.4.1)

Figures 61 and 62 must be used consecutively. Generally, two trials as illustrated in Figure 65 are enough to obtain the end results which are as follows for the 3.5 x 5 m drift.

Minimum length: 2.5 m
Maximum spacing: 1.25 m
Minimum support pressure: 0.04 MPa

A check should then be made that rock bolts used on the recommended spacing can provide the required minimum support pressure. This can be done through equation for calculation of bolt spacing of Section 4.4.2:

$$S = \left(\frac{60.10^{-3}}{0.04}\right)^{1/2} = 1.22 \text{ m} \approx 1.25 \text{ m}$$

Since the value of 1.22 m is almost equal to the assumed spacing of 1.25 m, the test can be accepted.

PARAMETER – EMPIRICAL RULES	TRIAL 1	TRIAL 2
Minimum length greatest of:		
A. Two times the bolt spacing	Undetermined	2 x 1.25 = 2.5 m
B. Three times the width of critical and potentially unstable blocks	NA	As before
C. For elements above the springline:		
1. Spans less than 6 m: one–half span	0.5 x 5 = 2.5 m	As before
2. Spans from 6 to 18 m: interpolate betwen 3 to 4.5 m lengths, respectively	NA	As before
3. Spans from 18 to 30 m: one–fourth span	NA	As before
D. For elements below the springline:		
1. For openings less than 18 m high, use lengths determined in C. (above)	NR (assumed)	As before
2. For openings greater than 18 m high: one–fifth the height	NR (assumed)	As before
Maximum spacing least of:		
A. One–half the bolt length	0.5 x 2.5 = 1.25 m	As before ← USE
B. One–and–one–half times the width of critical and potentially unstable rock blocks	NA	As before
C. 1.8 m	1.8 m	As before
Minimum average support pressure at yield point of elements greatest of:		
A. For elements above springline:		
1. Pressure equal to a vertical rock load of 0.2 times width of opening	0.2 x 5 x 0.026 = 0.026 MPa	As before
2. 0.04 MPa	0.04 MPa	As before ← USE
B. For elements below springline:		
1. Pressure equal to a vertical rock load of 0.1 times the opening height	NR (assumed)	As before
2. 0.04 MPa		
C. For elements at intersections, twice the greatest support pressure	NR (assumed)	As before
determined in A. or B. (above)	NR	As before

NA – not applicable; NR – not required

Fig. 65 – Example of rockbolt parameter determination with the U.S. Corps of Engineers' method

c) Farmer and Shelton's method (Section 4.4.4.2)

Rockbolt design is made by using Figure 63 in the following categories: excavation span less than 15 m, number of discontinuity sets equals two (average rock mass), inclined at 0–45° to horizontal. Rockbolt parameters provided by the table are the following.

$L = 0.3 \times 5$ m (span) $= 1.5$ m (bolt length)

$S = 0.5 \times 1.5 = 0.75$ m (bolt spacing)

d) Method of the minimum bolting densities surveyed in the Canadian Shield (Section 4.4.3)

$D = -0.227 \ln 10 + 0.839 = 0.32$ bolts per square meter

$$S = \frac{1}{0.32^{1/2}} = 1.77 \text{ m (bolt spacing)}$$

Figure 66 shows the end result of rockbolt parameters determination with the four methods applied to the two types of openings in the three different rock masses. Figures 67, 68 and 69 illustrate the length (L) and spacing (S) of Figure 66 with scaled sketches of the 20 x 15 m (65.5 x 49 ft) chamber and the 3.5 x 5 m (11.5 x 16.4 ft) gallery for each of the weak, average and strong rock types.

4.4.6 Discussion on Methods Used

Among the four methods, the NGI method is certainly the most elaborate. It seems to account very well for the three different types of rock, e.g. weak, average and strong, which were used for the worked examples in Figure 66. The other methods, especially the two empirical ones, are less accurate since they do not differentiate between the support to be used in the various types of grounds.

Comparing the NGI and the U.S. Corps of Engineers methods, it seems that the latter provides recommendations which apply essentially to average quality rock types. Actually, the two methods lead to similar length and spacing values of rockbolts for the 3.5 x 5 m (11.5 x 16.4 ft) gallery in this rock type, while a heavier support is recommended by the U.S. Corps of Engineers method for the 20 x 15 m (65.5 x 49 ft) chamber. The spacing value of 1.06 m is intermediate between the values of 0.68 m and 1.53 m of the NGI method for the weak and average rock types respectively.

			NGI'S METHOD			U.S. CORPS OF ENGINEERS METHOD			FARMER AND SHELTON'S METHOD			MINIMUM BOLTING DENSITIES IN THE CANADIAN SHIELD		
			Weak rock	Average rock	Strong rock	Weak rock	Average rock	Strong rock	Weak rock	Average rock	Strong rock	Weak rock	Average rock	Strong rock
3.5 x 5m (11.5 x 16.4 ft) **Drift**	ROOF	Length (m)	2.45	2.45	2.45	2.5	2.5	2.5	See	1.5	1.5	NS	NS	NS
		Spacing (m)	0.56	1.25	2.74*	1.25	1.25	1.25	Fig. 63	0.75	0.75	1.1	1.77	Unsupported
		Support pressure (MPa)	0.193	0.038	0.008	0.04	0.04	0.04	NS	NS	NS	0.82**	0.32**	0
	WALL	Length (m)	2.45	No wall support	No wall support	2.5	2.5	2.5	See Fig. 63			Same as roof if required		
		Spacing (m)	0.80			Same as roof if required								
		Support pressure (MPa)	0.142	0.028	0.005	0.04	0.04	0.04	NS	NS	NS			
20 x 15 m (69.5 x 49 ft) **Chamber**	ROOF	Length (m)	3.35	3.35	3.35	4.2	4.2	4.2	See	5	5	NS	NS	NS
		Spacing (m)	0.68	1.53	3.35*	1.06	1.06	1.06	Fig. 63	2.5	2.5	1.1	1.77	Unsupported
		Support pressure (MPa)	0.193	0.038	0.008	0.08	0.08	0.08	NS	NS	NS	0.82**	0.32**	0
	WALL	Length (m)	3.35	No wall support	No wall support	4	4	4	See Fig. 63			Same as roof if required		
		Spacing (m)	0.80			Same as roof if required								
		Support pressure (MPa)	0.142	0.028	0.005	0.04	0.04	0.04	NS	NS	NS			

* equivalent to spot bolting; **bolts per square meter; NS – not specified

Fig. 66 – Rockbolt parameter determination for two types of opening according to four methods

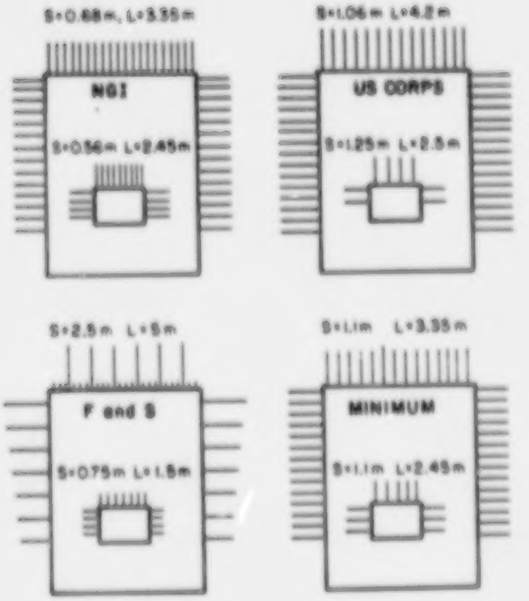

S=0.68m, L=3.35m
NGI
S=0.56m L=2.45m
S=1.06m L=4.2m
US CORPS
S=1.25m L=2.5m
S=2.5m L=5m
F and S
S=0.75m L=1.5m
S=1.1m L=3.35m
MINIMUM
S=1.1m L=2.45m

Fig. 67 – Illustration of calculated length and spacing of rockbolts for a chamber and a gallery in a weak rock type

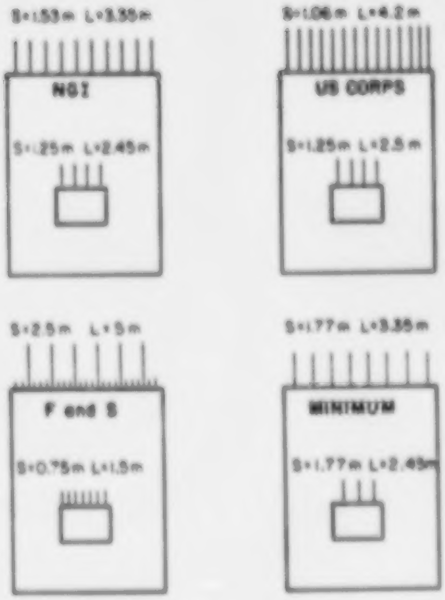

S=1.53m L=3.35m
NGI
S=1.25m L=2.45m
S=1.06m L=4.2m
US CORPS
S=1.25m L=2.5m
S=2.5m L=5m
F and S
S=0.75m L=1.5m
S=1.77m L=3.35m
MINIMUM
S=1.77m L=2.45m

Fig. 68 – Illustration of calculated length and spacing of rockbolts for a chamber and a gallery in an average rock type

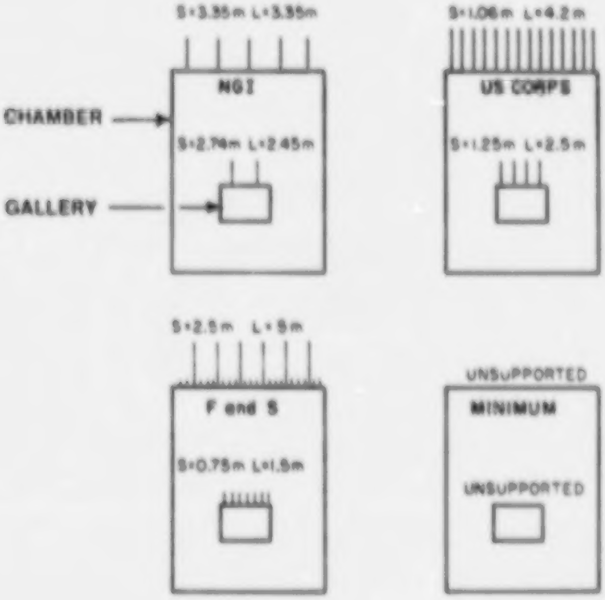

S=3.35m L=3.35m
NGI
CHAMBER
S=2.74m L=2.45m
GALLERY
S=1.06m L=4.2m
US CORPS
S=1.25m L=2.5m
S=2.5m L=5m
F and S
S=0.75m L=1.5m
UNSUPPORTED
MINIMUM
UNSUPPORTED

Fig. 69 – Illustration of calculated length and spacing of rockbolts for a chamber and a gallery in a strong rock type

The Farmer and Shelton rules give support recommendations which differ from the other methods. The spacing values obtained are unrealistic when compared to the values of the three other methods, being too close in the case of the drift, and too wide in the case of the chamber. Some useful information can be obtained from the set of rules, however, it is recommended that they should not be used as a sole method of rockbolt pattern design.

The method of Minimum Bolting Densities in the Canadian Shield, although incomplete since it provides only rockbolt spacing values, appears to be in accordance with the NGI method, since the same trends are observed. Actually, spacing values recommended are always wider than those of the NGI method, which is the objective of the method. As stated in Section 4.4.3, the method provides minimum bolting densities, or maximum bolt spacing values recorded in the mines of the Canadian Shield during the survey. These spacing values represent a minimum level of satisfaction, probably between support costs and minimization of accident risks, which were found in every mine.

4.4.7 Recommendations for Rockbolt Length and Spacing Selection

Two different approaches will be recommended. The first approach will be useful where no detailed rock mass investigation has been made. In this case, the U.S. Corps of Engineers method, which does not differentiate for rock types, should be used. It will provide length and spacing values that are realistic and that reflect already existing practices. As discussed in the previous section, the method applies more to average rock types, but it will be left to the designer to decide whether he or she is facing such rock conditions. In good and very good rock conditions, the method will lead to some overdesign. On the other hand, where the rock conditions are weak, the method should be dropped and the second approach should be used.

The second approach consists in evaluating the NGI geomechanics classification of the rock mass in which the opening will be excavated. At this point, it is useful to remember that the classification allows the rating not only of the quality of the rock mass but also of various other situations which can affect rockbolt design. For example, depth, presence of shear zones, rockbursting potential and other parameters, can be accounted for in the final rating of the rock mass classification.

The use of the two design methods is recommended: Minimum Rockbolting Densities in the Canadian Shield and NGI, one after the other. The first one will provide a maximum rockbolt spacing value which should not be exceeded, while the second method will refine the design to length and spacing values more in accordance with past practices which helped to develop the method.

4.4.8 Choosing the Arrangement of Bolts

The arrangement of bolts should generally be chosen so that bolts intersect the joints in the ground at an angle of more than 45° while being also installed at an angle of less than 10° in the wall of the gallery (Fig. 70). This arrangement is very important for mechanically anchored bolts, for which there is considerable loss of tension beyond this value of 10° with respect to the wall. In some cases, it may be very difficult to comply with these two conditions at the same time, and this makes it necessary to arrive at an acceptable compromise, either by using bevelled washers (Section 3.1.3) or by using fully anchored bolts, in which case it is less important to install them at an angle with the wall of less than 10°.

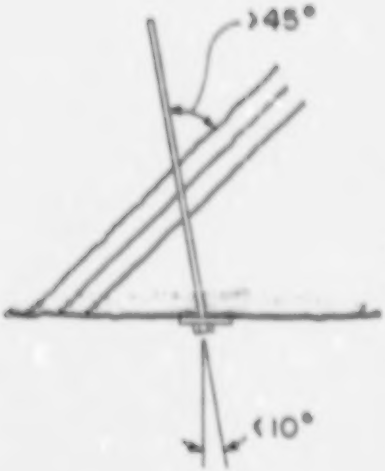


Fig. 70 – Angles that must be maintained during the installation of the bolts

Figure 71 shows various systematic bolting patterns in jointed and fissured grounds which meet the joint interception requirement discussed (an angle of more than 45°).

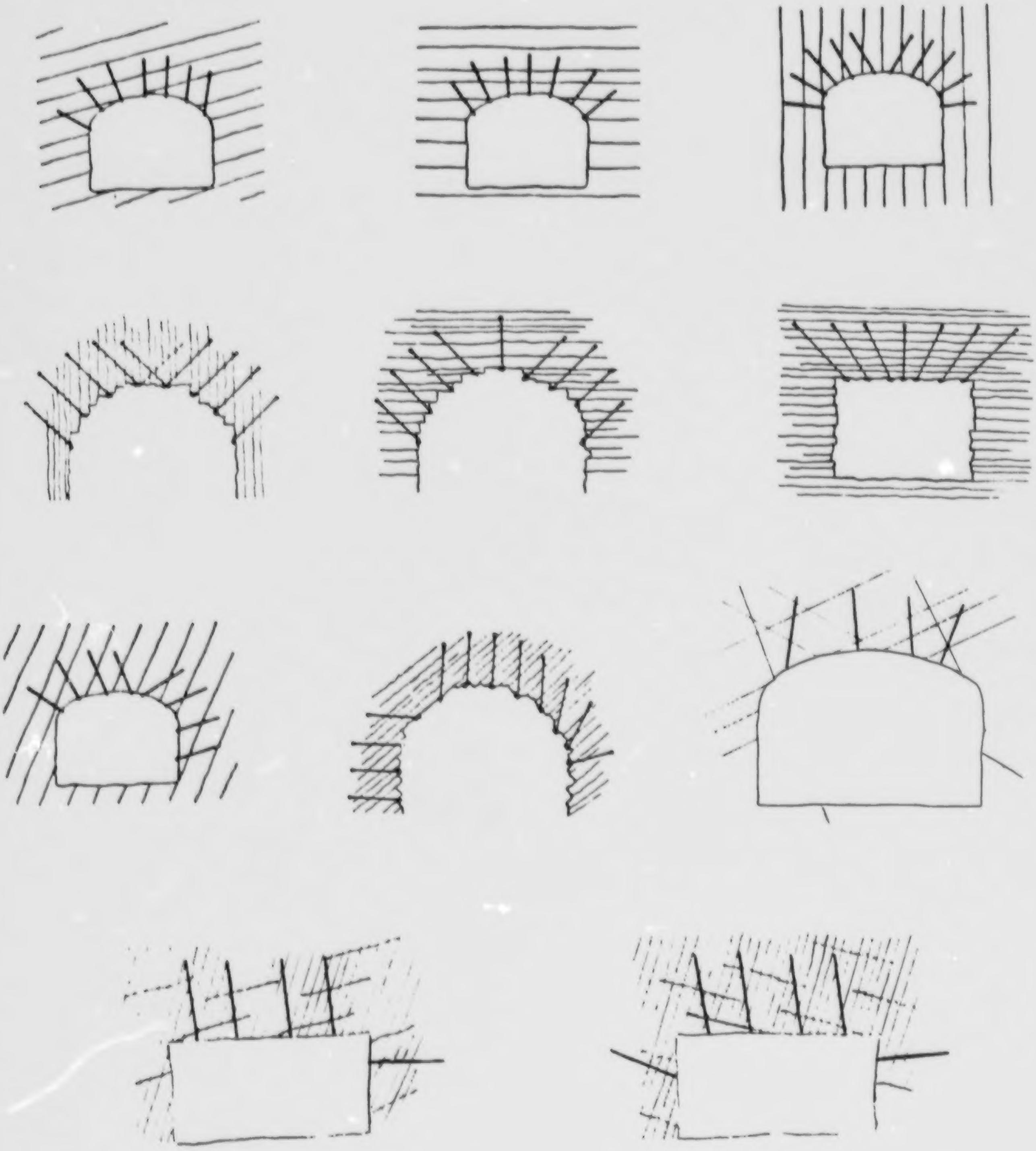


Fig. 71 – Systematic bolting patterns in fissured ground

Chapter 5: MECHANIZED ROOFBOLTING

This chapter outlines the principles of mechanized roofbolting and suggests improvements for the installation of various bolt types.

Chapter 5:
MECHANIZED ROOFBOLTING*

5.1 PRINCIPLES OF MECHANIZED ROOFBOLTING

In the mining environment, rockbolting is synonymous with safety, and to maintain safety, one must consider the method of installation as well as the bolt type or pattern. As bolts are installed in areas where rock falls are a potential hazard, the installation of roofbolts must be carried out under hazardous conditions. Therefore, any system that allows a machine to perform where a worker would be endangered is beneficial to the industry.

A major user of mechanized roofbolting equipment is the American coal mining industry, which has literally hundreds of semiautomated bolters in use. These bolters, while alleviating the hazard, do not eliminate it. Although the worker is not in the danger zone while drilling takes place, at the end of the drilling cycle he has to remove the drill steel and replace it with the bolt to be installed.

These semiautomated bolters use the rotary drill for torquing expansion shells or spinning the resin type bolts. Well aware of the problems this situation creates, the U.S. Mines Safety and Health Administration (MSHA) has recently introduced legislation which requires that an advanced roof support be mounted on all bolters of this type.

These units are further limited in application because they have been designed as rotary drills for the coals and shales of the U.S.A. and thus have no application in most Canadian hardrock mines. Furthermore, because these units employ the drill as a rotary device to install the bolts, they can only be used with expansion shell and resin type bolting systems.

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*This chapter was prepared in collaboration with Ivan E. Hugo, International Support Manager, Eimco Secoma.

There are, however, mechanized roofbolters available for hardrock mining which overcome these problems. This type of unit is not new to the hardrock mining market, the first one having been designed by Secoma (now known as Eimco Secoma) of France in the mid-1950s. Its patent lapsed in the early 1970s and mechanized bolters are now available from the other major hydraulic drill suppliers, Atlas Copco and Tamrock. Mechanized bolters are also now available from the smaller mining equipment suppliers, and most of these roofbolters use the turret or pivoting system originally developed by Secoma.

These units are able to drill the hard rock using hydraulic rotary percussive drills, which are both faster and more efficient than either jack hammers or pneumatic, rig-mounted drills. They also have the operator's compartment at least 6.1 m (20ft) from the actual drilling-danger zone (Figure 72). These units are normally fitted with canopies and the operator need not leave his compartment throughout the drilling and installation operation.

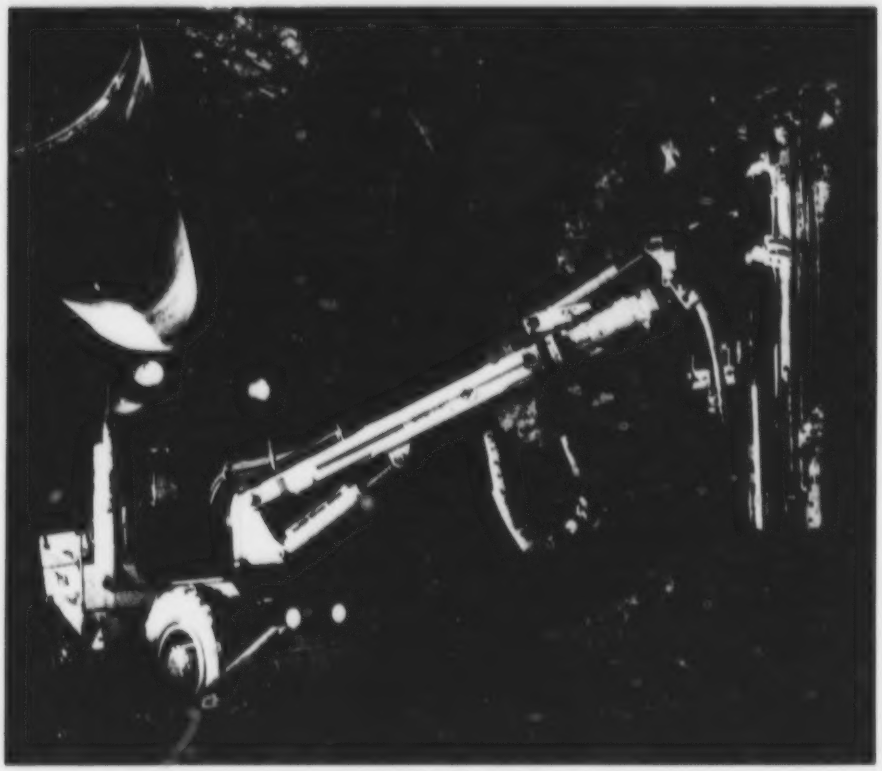

Fig. 72- Automated roofbolter with operator at a safe distance from operation. The boom can extend 1.7 m on this unit

The roofbolt magazine (Figure 73), which allows for mechanized replacement of roofbolts in the turret on completion of the bolting cycle brought increased safety. This system enables the operator to install a series of bolts without having to leave the operator's compartment. The capacity of these magazines (or carousels) varies with manufacturer and bolt and plate size. However the industry yardstick is a unit for resin bolts using 102 x 102 mm (4" x 4") plates and having a 13-bolt capacity.

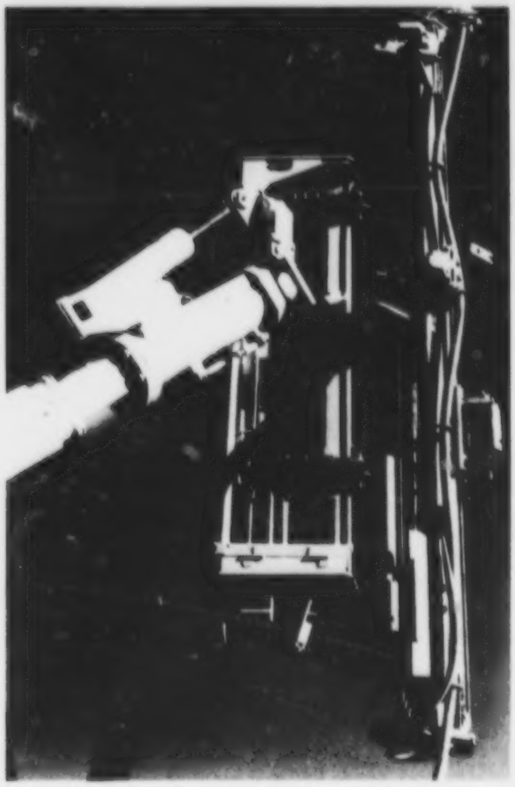

Fig. 73 – Roof bolt turret with bolt magazine

The unit is stationed under supported ground, with the bolting head so placed that the first bolt can be installed without boom extension. On installation, the boom is extended to the next position and the cycle repeated until the full magazine capacity has been installed. The boom is then retracted and only then does the operator leave the safety of his compartment. When he does move to replenish the bolts in the carousel, this can now be done under presupported ground, i.e., once a bolting pattern has been installed. This system, therefore, obviates the use of the advanced support required by MSHA on coal type bolting units.

As mentioned earlier, the most common system used on these mechanized bolters is the revolving turret principle (Figure 74).

Step 1. The turret is positioned against the roof.

Step 2. The hole is drilled.

Step 3. The drill is retracted and, when required, resin or cement is injected. The turret then revolves and a separate feed line introduces the bolt and torques or spins according to roofbolt type, without the anchor point ever having left its original position against the roof.

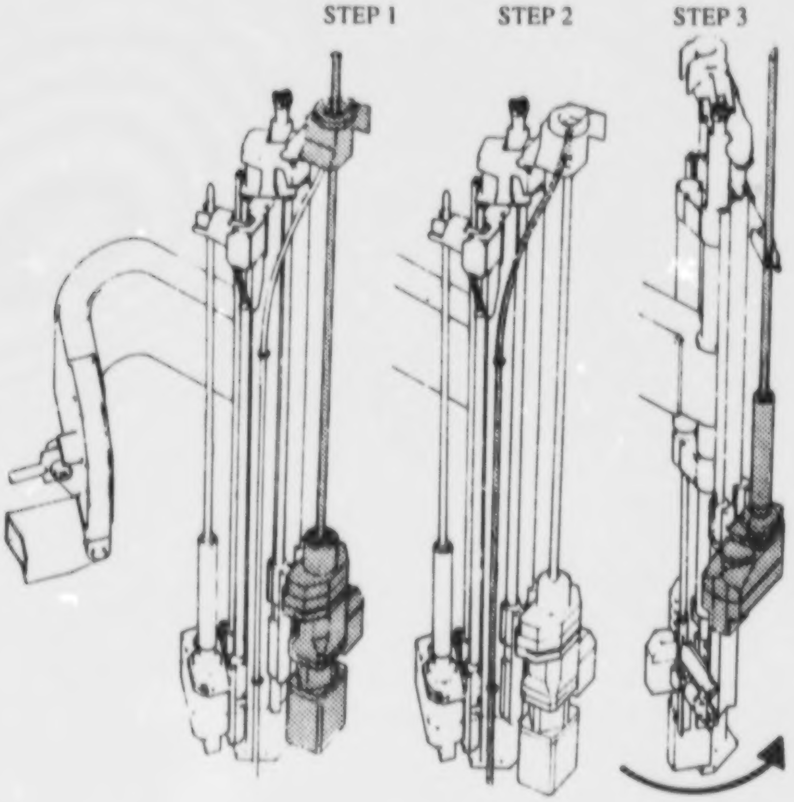

Fig. 74 – Revolving turret principle

An often unperceived advantage of mechanized bolting is that a far more accurate positioning of the bolt can be achieved because the effort of bolting and the danger of falling rock have been eliminated. This allows for more efficient support and reduces the number of actual bolts to be installed. Mechanization facilitates the installation of angled or inclined bolts.

particularly when radial patterns are called for. With mines becoming progressively deeper and mining conditions deteriorating as a consequence, the need for the installation of sidewall bolts and even floor pinning is also becoming greater.

Many studies and papers have been presented on the economics of different types of bolting systems, comparing the manual, semiautomated and fully automated approaches. However, the major benefit of mechanization is, and will always remain, the safety of the operator and the efficiency of the bolts installed. None of the studies really compares the effect of a badly installed bolt, and in the preceding chapters, results achieved have all been dependent upon bolts being properly installed. However, if one assumes that all bolts are installed with equal efficiency, the figures as presented in the recent study by Stillborg (9, pp. 126–134) are an accurate and valid indication of the comparative cost of the various systems.

Finally, it should be mentioned that manufacturers are generally well positioned to custom-build special units aimed at specific uses. Figure 75 illustrates, as an example, a bolter for 3.6-m (12-ft) long mechanically anchored bolts.

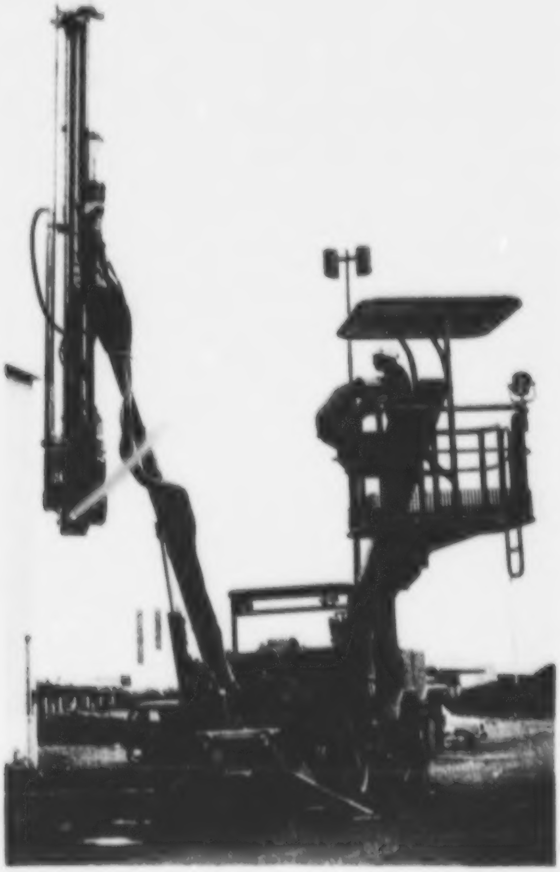

Fig. 75 – Roofbolter for 3.6-m (12-ft) long rockbolts

5.2 QUALITY OF BOLT INSTALLATION AND MECHANIZATION OF THE BOLTING PROCESS

The value of installing a bolt correctly is inestimable. An incorrectly installed bolt can constitute a greater danger than an area that is unbolted, because, once a hole has been drilled into virgin rock, the integrity of the rock mass (or beam) is compromised. In fact, a line of poorly installed bolts across a drift weakens the material.

To understand how mechanization can affect each of the various bolt types, one needs to look at these individually:

5.2.1 Mechanically anchored bolts

In manual bolting, the bolt holes are normally drilled using a pneumatic stoper mounted on a jack leg. This makes accurate positioning of bolt holes extremely difficult, and holes tend to be drilled in concave sections, where it is easy to collar the starter. This concavity then makes it extremely difficult to get good face contact between the plate and the rock surface. As a mechanically anchored bolt relies primarily on the head plate for support, this could severely compromise the bolt's integrity. This problem and that of inclined holes has been alleviated somewhat by plates with domes and slotted holes (Section 3.1.3).

Where stopers are used to drill the bolt holes, these are normally used for the tensioning of the expansion shell. This could have a detrimental effect on the integrity of the contact between the expanding shell and the face of the rock. The same caution applies to the use of impact wrenches for the tightening of expansion shells. However, as mechanized bolters use separate drilling and bolting lines, a simple rotary device giving a predetermined torque can be used to tension the expansion shell and the rock face.

The torque range is also much higher than that achieved with the rifle bar rotation of stopers. In fact, in certain cases, MSHA will give exemption from the torque testing of bolts if they are installed with an automated bolter with pre-set bolt torque.

5.2.2 Resin-grouted bolts

The efficiency of any resin bolt is determined by the efficiency of mixing of the resins and catalyst. Overspinning and underspinning can be equally dangerous - overspinning because the bond between the bolt being installed and the resin could be compromised if the bolt is spun with too high a speed and torque, and underspinning because if the bolt is not spun fast or long enough, the resin catalyst might not be mixed enough to ensure the optimum bond (Section 3.2.4.2). The latter case is particularly prevalent in areas where stopers with rifle bar rotation are used for spinning the bolt. Most mines work on a bonus system for the installation of roofbolts, which results in the tendency of the installing crew to rush through the spinning phase.

Due again to the separate drilling and bolting feed systems normally applied with mechanized bolters, the revolution speed (RPM) and torque of the spinning device can be adjusted according to resin type, as can the relationship between the rotation speed of the bolt and the feed advance, assuring not only that the resin is adequately mixed, but also that mixing takes place through the full column of resin and catalyst.

Using a manual system, one is forced to use an adaptor or a domed-head nut to prevent the nut riding up the bolt thread prematurely. Using the mechanized bolting system, one has the flexibility of changing RPM and the direction as required and this does nos apply. One of the major suppliers of automated roofbolters uses a bidirectional rotation system to overcome this problem. The bolt is rotated at high speed counterclockwise until the resin is properly mixed. Once the point anchor resin has hardened, the nut is then rotated clockwise at lower speed but higher torque to tension the plate against the roof. This change from high RPM, low torque to low RPM, high torque is achieved by using a simple sequence valve which operates as a clutch stalling the bolt once the point anchor resin has hardened, before switching the motor to the alternate direction and/or RPM setting.

One of the major disadvantages of mechanized resin bolting was the use of encapsulated resin, which had a very short shelf life and a high wastage factor due to inaccurate alignment of the resin injection line with the pre-drilled hole. To counter the misalignment, the earlier models of automated roobolters used a reamer to form a cone at the hole base. However, in the improved models of automated bolters available today, this is no longer the case. For instance, in the Eimco Secoma sliding head system, there is no movement of the turret between the drilling and the resin injection phases and the misalignment problem is totally eliminated. The resin manufacturers in turn have improved the products to such a degree that the shelf life is no longer a problem.

5.2.3 Cement-grouted bolts:

Many advantages of the full column resin-grouted bolts are negated by the cost. However, some of the properties of the resin bolt can be found in concrete-imbedded bolts, which do not have this cost disadvantage. There remains the greatest disadvantage of cement bolts, which is the curing time; there is no support during the curing period, when the bonding to the sides of the hole and the bolt is taking place. If the bolt moves while the concrete is hardening, the bond between bolts and concrete may be compromised. This problem is exacerbated by blasting in close proximity to the bolt pattern.

To compensate for the lack of support during curing, many different methods, such as accelerators, cement cartridges and fibrecrete, have been tried. All have their diverse advantages and disadvantages, but none have yet proved viable for general mining conditions. Therefore, concrete bolting has tended to be limited to the rehabilitation of haulage drifts and also with the cable bolts used in sub-level stoping and similar mining methods.

The quality and consistency of the cement must also be considered. Unlike resin, which is premixed and quality controlled, the quality of the cement bolt, where bulk cement is used as opposed to capsulated cement, depends on the operator and the method of mixing. If the system of encapsulated cement is used, the major cost advantage of cement over resin is lost. Therefore, if cement is to become a viable grouting medium, the costs need to be as low as possible, which indicates using bulk cement.

The three major roofbolt manufacturers, Atlas Copco, Tamrock and Eimco Secoma, have addressed this problem in various ways.

Atlas Copco has opted for a system similar to that of shotcrete pumps, whereby the dry cement powder and water are injected through several nozzles into the hole. While this system seems to overcome most of the problems, there is always inherent danger of the water to cement ratio changing due to system losses. The dust generated in this system, like that of shotcrete is normally cement powder, indicating that there is not 100 percent mixing of the atomized water and dry cement.

Tamrock uses a bulk mixing system mounted on the unit, where the cement is constantly agitated to avoid hardening (a similar system to that used with bulk cement transporters). This system does overcome most problems, and the only change in consistency of the cement is the natural drying caused by constant agitation. However, for this system to work, the cement grout itself has to be fairly fluid, and the tendency is to keep it wetter than normally required, not only to facilitate pumping, but also for cleaning at the end of the shift.

Eimco Secoma uses a system of mixing dry cement with a fixed quantity of water on demand, i.e., the dry cement and water are only mixed when pumping takes place. This system has the advantage of not producing large quantities of mixed cement which could harden and cause problems when cleaning is required. The consistency of the cement can be adjusted and, where optimum consistency is required, it will remain constant. These combined advantages allow the cement to be drier than bulk mixes. This allows for higher final compressive strength of the grout and increases the bonding characteristics of the bolt in the borehole (Sections 3.2.5.1. and 3.2.5.2.).

Besides the low cost of the cement required for full-column cement-grouted bolts, the major saving achieved for this type of bolt lies in the bolt itself. As no spinning is required and the support is achieved along with full length of the bolt, plain rebar can be used, cut to length without any forging of heads or plates. If, however, head plates are required, the type of plate used for areas where second plates are installed to hold mesh can be used.

These advances have resulted in machines that can install more than one type of bolt - an immediate-acting bolt such as a resin, split set or swellex for areas where immediate support is required, and concrete bolts where the longer curing time is acceptable.

5.2.4 Split Set bolting

Split Set bolts lend themselves to manual installation, in that a normal stoper can be used to drill a hole and then, using an adaptor, to install the bolt itself. However, in practice some difficulties are encountered, especially when longer bolts are used, with the bolts bending due to the lack of adjustment to blow energy or frequency on pneumatic hammers. However, the new mechanized bolters tend to use hydraulic hammers, where both the feed force to push the bolt into the hole and the impact can be adjusted according to the environment, facilitating bolt installation. An example is the Eimco Secoma HH200 impactor for installing Split Set bolts. The HH200 has three different blow settings and is totally compatible with the Hydrastar 200 hydraulic drifter (manufactured by Eimco Secoma), which is normally used on the bolters for drilling the hole. Variable feed force and speed aid the insertion of the Split

Set and are easily controlled on mechanized bolters using the relief valves and flow control valves normally in the system for drilling.

5.2.5 Swellex bolting

Swellex bolts lend themselves more easily to manual installation than most other types; however, for reasons of the operator's safety, mechanized bolting is also advised in this case. The Swellex system for mounting on roofbolters is available from all the major suppliers and can quickly and easily be mounted on most standard bolting units.

5.3 MESHING

The one area where mechanization has yet to establish itself as being viable is the area of meshing. In Canada, there have been approaches to companies to provide a machine for this aspect of underground support, but to date no notable successes have been achieved.

It is difficult to mechanize the cutting of rolls of mesh because the cutter must move across the full width of the roll and the consequent sharp edges present a danger. When pre-cut sheets are used, these need to be stockpiled, and here one is faced with the difficulty of mechanically taking them from the stock to the working position.

While neither of these problems are insurmountable, it appears that a viable solution is not yet commercially available. Discussions with equipment manufacturers indicate that it might be best to look at the two operations, bolting and meshing, in isolation at present. More effective bolting could well diminish scaling and the subsequent need for meshing.

Chapter 6: CONCLUSIONS

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To prepare the *Rock Bolting Practical Guide*, we attended numerous meetings and carried out literature searches, tests, and field surveys. This made it possible to gather in a single document a large amount of information on the performance of various types of rock bolts, as well as on methods used to select their characteristics. Test results contributed by many involved in rock bolting (manufacturers, mining companies, research centres, government departments) are discussed throughout the text.

Nevertheless, on reading this guide, it is obvious that, although many points regarding the performance of bolts have already been investigated thoroughly, there are also many for which we have woefully inadequate or no information.

The following list was prepared with this in mind. It represents suggestions on a range of points, all of which are pertinent to rock bolting in Quebec mines. These suggestions should be investigated by one or more of the previously mentioned participants in the mining industry. For each of these points we have indicated the reference section in the guide.

1) Choose several mines that are representative of the principal ground conditions encountered in the Canadian Shield (soft volcanic rocks with vertical schistosity, hard and fissured intrusive rocks, soft and fissured intrusive rocks). Choose several gallery sections in each mine to install four types of bolts over sections 15 m long. The bolts would be equipped with load cells (mechanically anchored bolts) or strain gauges (fully anchored bolts, Section 3.2.7.1) to determine the tension developed in them as a function of time. Moreover, internal ground-expansion measures on the roof of the galleries should be obtained using extensometers with multiple anchors slightly longer than the bolts.

The purpose of this testing would be to compare the performances of each type of bolt under identical ground conditions. It would also demonstrate the formation of natural rock vaults contributing to the stability of the gallery roof, as well as the eventual formation of loose zones immediately surrounding the gallery roof, where the purpose of bolting would be to keep the blocks of rock in place to prevent loosening (Section 2.3.1).

2) Install two series of mechanically anchored bolts tilted at an angle of 10° to the wall with head plates measuring 102 x 102 x 6.4 mm (4" x4" x 1/4") and 102 x 102 x 9.5 mm (4" x 4" x 3/8"), to compare long-term tension losses caused by each type of plate (Section 3.1.3). Loss of tension would be measured using a torque roof bolt tensioner (Appendix 6 and Figure 14).

3) Install two other series of mechanically anchored bolts tilted at an angle of 15° (or more) to the wall with semi-spherical and bevelled washers (Section 3.1.3) to establish whether they cause long-term loss of tension in the bolts.

4) Conduct a series of tightening tests using a torque wrench to establish the relationship between tension and installed load torque for the FH bolts used with and without HSW, and then with and without a wire mesh and wooden washer. The tests discussed in Section 3.1.4 and Appendix 7 were mainly carried out using bolts threaded at both ends, which are more commonly used in Ontario than in Quebec mines. The purpose of this test would be to confirm that the use of HSWs produces greater uniformity in the initial tension of the bolts after tightening.

5) Install FH bolts equipped with HSWs at an angle of more than 10° to the wall and drive them in as tightly as possible. The purpose of this test would be confirm the statement made in Section 3.1.4, that it is possible to observe shear fracture in the heads of bolts installed under these conditions. It is because of this risk that in Section 3.1.4 we do not recommend the use HSWs with FH bolts.

6) Carry out installed load tests using No. 9: 3.75 mm (0.148") welded wire meshes measuring 102 x 102 mm (4" x 4") with wooden washer and a 102 x 102 mm (4" x 4") head plate, and then without a wooden washer and with 127 x 127 mm (5" x 5") and 152 x 152 mm (6" x 6") head plates. The purpose of these tests would be to verify whether the same load-bearing capacities can be obtained in both cases (Section 3.1.7 and Appendix 8). If both cases can bear the same load, then wider head plates should be used instead of a wooden washer, which causes a certain long-term loss tension in the bolts (Section 3.1.5).

7) Carry out a series of installed load torque measures on the bolts, both when they are installed at the gallery heading and at regular intervals thereafter. Some bolts should be tightened using 230 N-m (170 lb-ft) and others using 162 N-m (120 lb-ft). The purpose

would be to confirm the results discussed in Section 3.1.8 and Appendix 10, which indicate that the loss of tension in the bolts is low after blasting vibrations when the initial installed load torque is high.

8) Repeat the pull-out tests on mechanically anchored bolts installed in drill holes that are slightly outside the +1.6 mm (+1"16") diameter tolerance recommended by the manufacturers of expansion shells. The purpose would be to confirm the results of the tests discussed in Section 3.1.9 and Appendix 11, which show that the holding power of the bolts is greatly diminished under such circumstances.

9) Carry out pull-out tests on mechanically anchored bolts tightened using 230 N-m (170 lb-ft), on bolts tightened using 230 N-m (170 lb-ft) and then loosened to 121 N-m (90 lb-ft) (to simulate the loss of tension because of blasting vibrations), and on bolts initially tightened using 121 N-m (90 lb-ft). The purpose would be to confirm the results discussed in Section 3.1.9, which show that the performance of the bolts is the same in the three cases, that is, that they would show the same load exceeding the yield strength of 9 tonnes and anchor slip in the vicinity of 1.6 mm per tonne (0.028" per 1,999 lb).

10) Carry out dynamic pull-out tests on mechanically anchored bolts tightened with torques of different values: 121, 162, 230 N-m (90, 120, 170 lb-ft), for example, by using a loader to let drop block of rock suspended by a chain from the bolt. The purpose would be to compare the performance of bolts in the three cases by simulating one of the most adverse rock fall conditions in the roof of a gallery.

11) Carry out pull-out tests of mechanically anchored bolts in sulphide rock mass formations to confirm the results discussed in Section 3.1.9 and Appendix 11, which show that the pull-out force is reduced from 124.4 kN (28,000 lb) to 97.8 kN (22,000 lb) under such conditions. Theses tests could lead to specific recommendations for bolting sulphide rock masses, for example, to recommend the use of a type R expansion shell in very hard sulphides (Section 3.1.2).

12) Carry out pull-out tests of resin-grouted bolts in hard rock (compressive strength over 100 MPa (14.500 psi)) because their adherence factors are not well known (Section 3.2.4.2) and have been verified mostly in soft rocks. These tests will make it possible to take into account the difference between the diameter of the drill hole and that of the bolt by choosing

two deviations, 6.4 mm and 9.5 mm (1/4" and 3/8"), because this parameter modifies the observed stiffness of the bolt (Section 3.2.4.1).

13) Carry out comparative pull-out tests of resin-grouted bolts to establish the performances of products made by manufacturers, since the composition of the resins may vary widely (Section 3.2.3).

14) Carry out comparative pull-out tests of bolts grouted with cement and cement cartridges (Sections 3.2 and 3.2.1.2), to establish the characteristics of these two types of grouting, and confirm that the holding power and adherence factors obtained are similar (and even higher in some cases) to those of resin-grouted bolts (Sections 3.2.4.2, 3.2.5.2, 3.2.5.3, and Appendix 15).

15) Install load cells under the head plates of resin-grouted bolts to confirm that they carry a load that increases over time. The purpose would be to confirm the advisability of using head plates with fully anchored bolts (Section 3.2.6).

16) Establish test sections with various types of support and support combinations (Split Set bolts and wire mesh, Split bolts and resin-grouted bolts, mechanically anchored bolts and Swellex bolts, etc.) in a mine subject to rock bursts (Section 3.4.3). The long-term objective would be to compare the performances of each type of bolt in sections of the gallery that would eventually be affected by rock bursts.

17) Keep in centralized data bank the results of all the bolt pull-out tests carried out in Quebec mines. The tests should be carried out in accordance with a standardized procedure (Section 3.1.9) and include a record of the load and elongation. One of the objectives of the recommended in Section 4.3.3. In some cases, this scale becomes a primary tool in choosing the type of bolt that is most suitable for the ground conditions under consideration (Section 4.3.2).

18) Draw up a survey of a large number of case studies of galleries and other wide unsupported mine openings in Quebec mines, to adapt the graphs to predict the support needs described in Section 4.4.1 for these mines.

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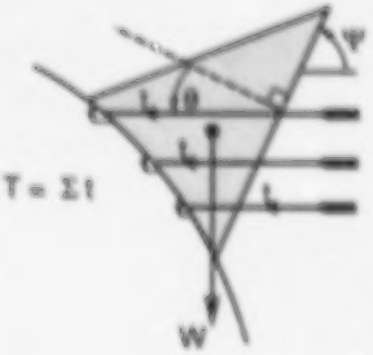
APPENDICES

A.1 – THE USE OF ROCK BOLTING TO STABILIZE A SLIDING BLOCK



The bolt tension required to stabilize a sliding block can be obtained using the following formula (A.1, p. 247–248*):

$$T = \frac{W[F.\sin\psi - \cos\psi.\tan\phi] - c.A}{\cos\theta\ .\tan\phi + F.\sin\phi}$$

where

T: sum of tensions in the bolts

W: weight of the block

A: area of the slip surface at the base of the block

ψ: slope of the slip surface

θ: angle between the bolts and a line perpendicular to the slip surface

c: cohesion along the slip surface

φ: angle of friction of the slip surface

F: safety factor

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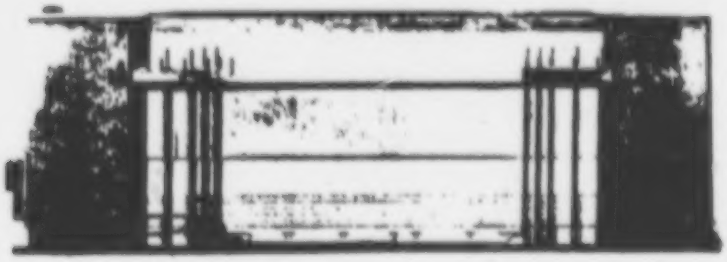
* The references are listed after the appendices.

A safety factor between 1.5 and 2 is chosen, depending on whether the bolts used are grouted over their entire length or are mechanically anchored, to take into account the possibility of loss of anchoring quality.

It is preferable to use tensioned bolts, to minimize displacements along the slip surface. If untensioned bolts are used, the safety factor used should equal 2, and the tension taken up by each bolt will be assumed to be 80% of the load exceeding the yield strength of the steel.

A.2 – DIMENSIONING GRAPH FOR TENSIONED BOLTS USED TO SUPPORT HORIZONTALLY BEDDED GROUNDS

At the beginning of the 1960s, Panek (A.2) carried out many tests on reduced plaster models of jointed gallery roofs reinforced by tensioned bolts (see illustration below). These models were placed in a centrifuge that made it possible to increase gravity artificially to simulate the true weight of the ground.



Reduced model of a gallery roof. From Panek (A.2)

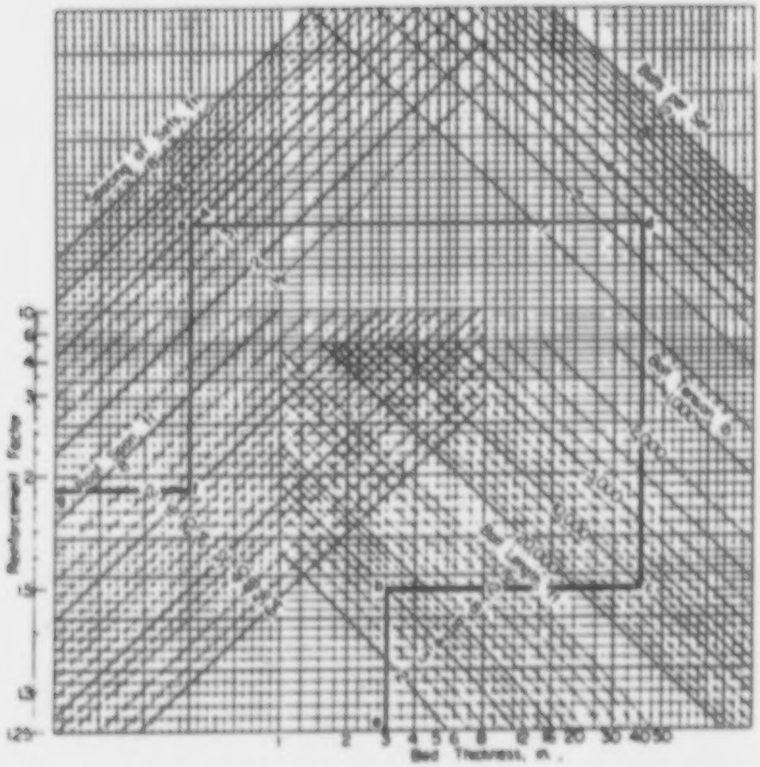
The author evaluated the beneficial effect of bolts by comparing the maximum stress caused by sagging in the centre of the roof in both the bolted (σ_{fb}) and non–bolted (σ_f) conditions. This led to the definition of a reinforcement factor (RF) that can be obtained as follows:

$$RF = \frac{\sigma_f}{\sigma_{fb}}$$

Through a series of regressions using the experimental results, a graph that can be used to evaluate the RF factor was developed (see illustration next page).

We should point out that this graph does not in itself guarantee the stability of the roof; rather, it makes it possible to evaluate the effectiveness of one rock bolting system in relation to another, or in relation to a situation where no bolts are used.

In general, a reinforcement factor of at least 2 is recommended for grounds where the spacing of the layers is not too pronounced (less than 15 cm), and a slightly lower factor for thicker layers.



Graph for determining the amount of tensioned bolt reinforcement needed for a bedded roof. From Panek (A.2)

On the graph, the example shown corresponds to strata 7.6 cm (3") thick (a), reinforced with 1.2 m (4 ft) bolts (b), tensioned to 44.4 kN (10,000 lb) (c), in rows of 3 bolts (d), with a spacing of 1.2 m (4 ft) (e), for the roof of a gallery 4.9 m (16 ft) wide (f). Under these conditions, the reinforcement factor (RF) is 1.9 (g).

A.3 – REDISTRIBUTION OF STRESSES AROUND UNDERGROUND EXCAVATIONS

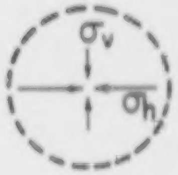
Before the excavation, the rock mass is under a vertical stress (σ_v) that is proportional to the depth, and a horizontal stress (σ_h) that is equal to a multiple of (σ_v).

Generally, these stresses are obtained as follows:

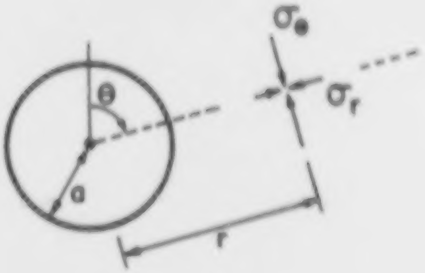
$$\sigma_v\ (MPa) = \gamma . h\ .\ .\ .\ .\ (A.3.1)$$

$$\sigma_h\ (MPa) = k_o\ .\ \sigma_v\ .\ .\ .\ .\ (A.3.2)$$

where	γ:	specific gravity of the rock (MN/m^3)
	h:	depth (m)
	k_0:	constant proper to the rock mass, often equal to 2.0 in grounds in the Canadian Shield



BEFORE EXCAVATION:



AFTER EXCAVATION:

After the excavation, the natural stresses (σ_v and σ_h) are redistributed around the periphery. In a simple case, such as in a circular gallery, the formulas used to calculate the stresses in the wall and inside the rock mass can be expressed as follows (A.1, p. 104):

$$\left|\begin{array}{ll} \sigma_\theta = \frac{1}{2} \cdot \sigma_v \left[(1 + k_o)\left(1 + \frac{a^2}{r^2}\right) - (1 - k_o)\left(1 + 3\,\frac{a^4}{r^4}\right)\cos 2\,\theta\right] & \ldots\ (A.3.3) \\ \sigma_r = \frac{1}{2} \cdot \sigma_v \left[(1 + k_o)\left(1 - \frac{a^2}{r^2}\right) + (1 - k_o)\left(1 - 4\,\frac{a^2}{r^2} + 3\,\frac{a^4}{r^4}\right)\cos 2\,\theta\right] & \ldots\ (A.3.4) \end{array}\right.$$

with
σ_θ: stress in the axis parallel to the wall
σ_r: stress in the axis perpendicular to the wall
a: radius of the circular gallery
r: distance from the centre of the circular gallery
θ: angle in relation to the vertical axis

It is interesting to determine the value of stresses σ_θ and σ_r in the periphery of the gallery, at the roof, and on the lateral walls. To do this, we assume that $r = a$ in both equations, and that $\theta = 0°$ for the roof, and $\theta = 90°$ for the walls:

In the roof: $$\left|\begin{array}{l} \sigma_\theta = (3\,k_o - 1).\sigma_v \\ \sigma_r = 0 \end{array}\right.$$

In the walls: $$\left|\begin{array}{l} \sigma_\theta = (3 - k_o).\sigma_v \\ \sigma_r = 0 \end{array}\right.$$

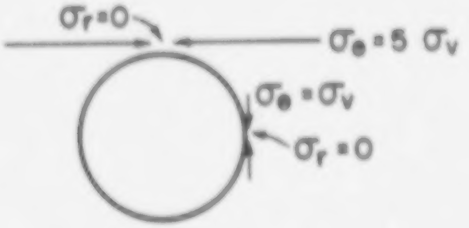
Then, if we let $k_0 = 2$, we obtain:

In the roof: $$\left|\begin{array}{l} \sigma_\theta = 5\ .\ \sigma_v \\ \sigma_r = 0 \end{array}\right.$$

In the walls: $$\left|\begin{array}{l} \sigma_\theta = \sigma_v \\ \sigma_r = 0 \end{array}\right.$$

Thus, in the wall, we now have the situation illustrated below:

STRESSES IN THE WALL:



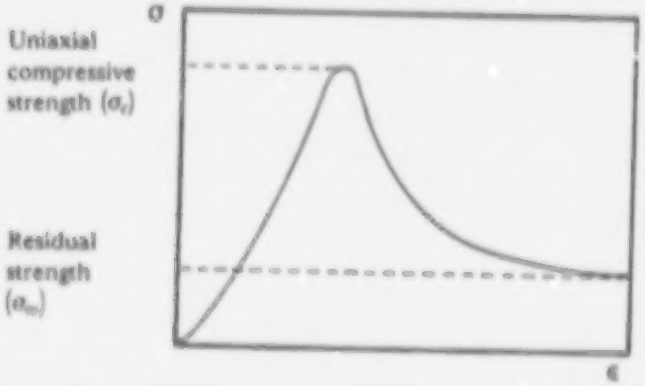
The tangential stress (σ_θ) in the roof is 5 times higher than the initial stress (σ_v) (or 2.5 times higher than the initial stress (σ_h) which was in the same direction); while stress (σ_r) has dropped to zero. In other words, the rock is no longer in a triaxial stress condition where it would have high strength, but rather in an uniaxial stress condition that will lead to failure if the value of (σ_θ) exceeds the uniaxial compressive strength.

In reality, the calculations show that stress (σ_r) is always equal to zero in the wall, regardless of the shape of the gallery; while stress (σ_θ) generally increases in relation to the value of the initial stresses (σ_v) and (σ_h). However, there are situations where the value of (σ_θ) decreases in relation to the initial stresses and may even become a tensile stress. These particular situations correspond to elongated mining stopes (where the height is at least three to four times greater than the width).

There are no analytic solutions that can be used to calculate the stresses in the wall of cavities with arbitrary shapes. To do this, we use stress analysis computer programs that employ the finite elements method, or the method of boundary elements. A program of the latter type is described by Hoek and Brown (A.1, p. 493–516), who used it to calculate the stresses around cavities of various shapes, and to produce graphs (A.1, p. 467–492) showing stress values that could, in many cases, be used as initial estimates.

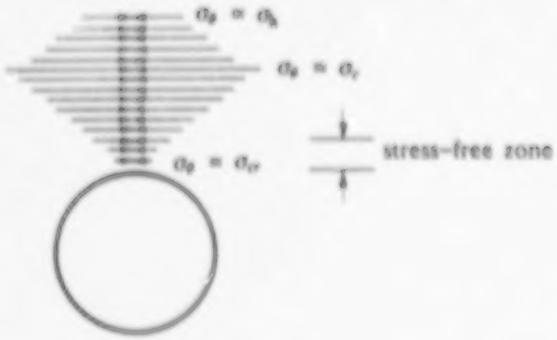
A.4 – STRESS-FREE ZONE AROUND UNDERGROUND EXCAVATIONS

As shown in Appendix 3, failure occurs in the wall of the cavity when stress (σ_θ) exceeds the value of the uniaxial compressive strength (σ_c). In this case, the rock cracks and the value of (σ_θ) decreases, because the rock enters a post-failure deformation phase that is clearly visible in the stress–strain diagram $(\sigma - \epsilon)$ below.



Some stress analysis programs make it possible to take into account the new stress distribution in the rock mass after the uniaxial compressive strength in the wall is exceeded. This distribution is such that the value of stress (σ_θ) is equal to (σ_{cr}) at the wall, gradually increases to the value of (σ_c) inside the rock mass, and finally decreases to the value of the initial stresses.

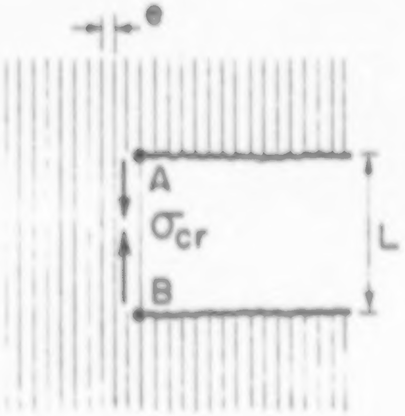
The zone in the periphery of the wall where the only stresses are the low (σ_θ) value is the stress-free zone. The presence of this type of zone in the walls of galleries and mining stopes, particularly when they are at a sufficient depth that the (σ_c) strength is exceeded, plays a major role in reinforcement. In fact, in this area, natural ground fissures are only slightly compressed by the (σ_θ) stress, and the blocks of rock bounded by these fissures are likely to slide or fall. Thus, the purpose of rock bolting is to prevent movement along these fissures and to participate in the formation of a natural rock vault that would be self-supporting.


$\sigma_\theta = \sigma_h$
$\sigma_\theta = \sigma_c$
$\sigma_\theta = \sigma_{cr}$
stress–free zone

Yu (A.3) is one of several investigators who have demonstrated the presence of a stress–free zone in the periphery of underground cavities. As we mentioned in Appendix 3, this zone may nevertheless appear only after a depth where the strength of the rock mass, or the joints that it contains, is exceeded.

A.5 - BOLTING SYSTEM REQUIRED TO STABILIZE THE BUCKLING OF SCHISTOUS LAYERS ON THE WALL OF A GALLERY OR STOPE

For the first layer embedded in the wall between A and B, the critical stress that causes buckling (also known as Euler's critical stress) can be obtained as follows (A.4):



$$\sigma_{cr} = \frac{\pi^2 . E . e^2}{3 . L^2} \qquad \ldots (A.5.1)$$

where e: thickness of the layers (m)

L: distance between clamping points A and B (m)

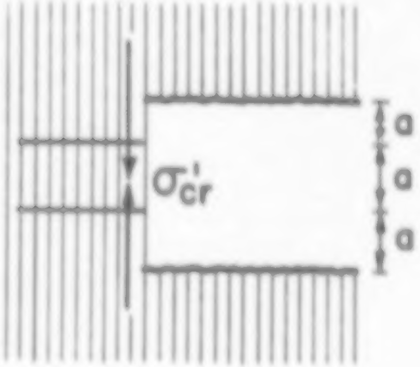
E: Young's modulus (MPa)

However, if we install N rows of bolts in the wall, the distance between the clamp points is reduced to the spacing of the bolts (a); assuming that they are placed at a distance (a) from the ends:

$$a = \frac{L}{N + 1} \qquad \ldots (A.5.2)$$

Thus, the critical stress becomes:

$$\sigma'_{cr} = \frac{\pi^2 . E . e^2}{3 . a^2} \qquad \ldots (A.5.3)$$

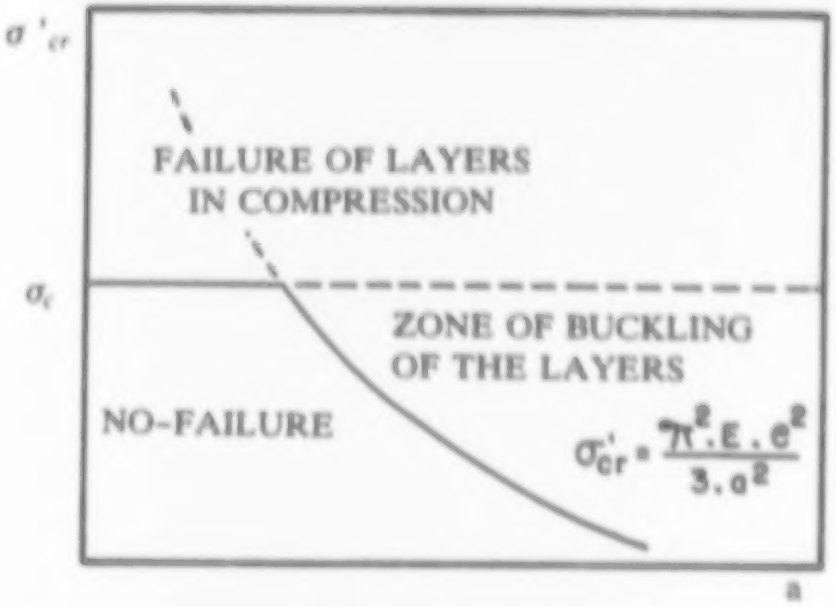


$$\sigma'_{cr} = \frac{\pi^2 . E . e^2}{3 . L^2} \, . \, (N + 1)^2 \qquad \ldots (A.5.4)$$

If we compare equations (A.5.1) and (A.5.4), we can see that the critical buckling stress increases by a factor equal to $(N + 1)^2$. For example, for N = 2, we have $\sigma'_{cr} = 9 . \sigma_{cr}$. In other words, with two rows of bolts, the tangential stress in the wall (see Appendix 3) that would be necessary to cause buckling of the layers is nine times higher than that which would cause it if there were no bolts. In this example, the use of rock bolting greatly increases the safety factor in the wall.

In a situation where we want to choose the optimal bolting pattern, we may suggest tracing the graph shown below with the help of formula (A.5.3). The graph also takes into account the value of the uniaxial compression strength (σ_c) which may become lower than the critical buckling stress, when the bolts are spaced relatively close together.

The dimensioning process consists of evaluating the value of the shear stress (σ_θ) in the wall (Appendix 3), and locating it on the ordinate of the graph. If (σ_θ) is higher than (σ_c), we are in an area where we can expect compressive failure, with the formation of a stress-free zone (Appendix 4), and bolting, even though it is necessary, will have a different effect. On the other hand, if (σ_θ) is lower than (σ_c), the distance (a) between the bolt rows can be selected on the graph to remain in the no-failure zone.



Typical values of (σ_c) and (E) for various types of rock are shown in Hoek and Brown (A.1, p. 141–142). Nevertheless, it would still be preferable to carry out uniaxial compressive tests with measures of deformations in the direction parallel to the schistosity.

A.6 – LOSS OF TENSION IN EXPANSION BOLTS USED WITH HEAD PLATES OF VARIOUS TYPES

Two series of tests are discussed.

a) Stelco Tests (A.5)

These tests were carried out on the block of granite used by the company to perform calibration tests on bolting materials.

They consisted of installing rock bolts with different expansion shells and equipped with domed head plates. The bolts were tightened with a torque wrench using torques from 202 N–m to 404 N–m (150 to 300 lb/ft), and torque readings were obtained during the subsequent four days.

The results obtained are shown in the table below, from which we can derive the following conclusions:

- Torque losses occur mainly during the first day and are completely stabilized by the fourth day.
- Torque losses are significant when the initial installed torque is close to 236 N–m (175 lb–ft). For higher values 270 to 404 N–m (200 to 300 lb–ft), the loss appears to be proportionally smaller.
- Recorded torque losses are attributable solely to the deformation of the plate and not to anchor slip. To confirm this effect, the same plates used for the first series of tests were re-used with other bolts and produced no loss of tension (see table below).

Bolt	Anchor	Installed load (lb–ft)	Loss after 1 day (lb–ft)
1	A	200	0
2	A	170	0
3	A	170	0
4	B	160	0
5	B	150	0
6	B	180	0
7	C	160	0
8	C	165	0
9	C	160	0

Torques measured with domed support plates							
Bolt	Anchor	Installed load torque (lb–ft)	Loss after 1 day (lb–ft)	Loss after 2 days (lb–ft)	Loss after 3 days (lb–ft)	Loss after 4 days (lb–ft)	Total loss (%)
1	A	150	0	5	5	5	3
2	B	150	10	15	20	20	13
3	C	160	30	40	40	40	25
4	A	175	55	75	75	75	43
5	A	175	20	20	20	20	11
6	A	175	25	25	25	35	20
7	B	175	20	30	30	30	17
8	B	175	0	5	5	5	3
9	B	175	0	0	0	0	0
10	C	190	70	70	70	70	37
11	A	200	10	20	20	20	10
12	B	230	0	30	30	30	13
13	C	250	5	10	20	30	12
14	C	300	50	65	65	65	22

Fig. A.1 – Results of torque loss tests with domed head plates.
From Lachapelle (A.5)

The torque loss observed with the domed plates seems to be the result of the deformation of the plates during the initial tightening. Contact irregularities between the plate and the support surface may also affect torque loss. Thus, retightening the bolts equipped with this type of anchor is extremely important and necessary.

Some flat head plates were tested in the same way and produced no torque losses. The results are shown below. However, given the small number of tests, supplementary studies should be carried out to confirm this trend.

Bolt	Anchor	Installed load (lb–ft)	Loss after 1 day (lb–ft)
1	–	190	0
2	–	200	0
3	–	210	0

b) Inco Tests (A.6)

Forty-eight double threaded (DT) mechanically anchored bolts were installed in an underground gallery with a stoper drill and driven in as tight as possible. The initial torque was measured with a torque wrench, while the tension in the bolt was measured during the first and subsequent days with a torque roof bolt tensioner.

The following combinations of head plate, plywood washer, wire mesh, and hardened steel washers (HSW) were tested:

1 = ▪ 102 x 102 x 6.4 mm (4" x 4" x 1/4") head plate

2 = ▣ head plate and plywood washer measuring 152 x 152 x12.7 mm (6" x 6" x 1/2")

3 = ▦ the same as 2. with 51 x 51 mm (2" x 2") no. 9 (3.76 mm (0.148")) welded wire mesh

4 = ▪ HSW the same as 1. with HSW

5 = ▣ HSW the same as 2. with HSW

6 = ▦ HSW the same as 3. with HSW

The results of these tests are summarized in the table in Figure A.2. and take into account the use of two types of expansion shells.

Type A Expansion Shell

Number of tests	Head Plate		Installed load torque (lb–ft)	Initial tension (tonnes)	Loss after 1 day (tonnes)	Loss after 2 days (tonnes)	Loss after 7 days (tonnes)	Loss after 15 days (tonnes)	Loss after 32 days (tonnes)	Loss after 104 days (tonnes)	Total loss (%)	Residual tension (tonnes)
4	▫		185	5.3	0.5	0.6	0.9	1.2	1.5	1.5	28	3.8
4	▣		206	4.0	0.3	0.6	1.1	1.3	1.5	1.8	45	2.2
4	▦		204	4.1	0.5	0.8	1.3	1.4	1.6	1.8	44	2.3
4	▫	HSW	179	5.6	0.1	0.4	0.5	0.6	0.8	0.9	16	4.7
4	▣	HSW	184	5.0	0.4	0.5	0.9	1.2	1.5	1.9	38	3.1
4	▦	HSW	178	5.1	0.2	0.3	0.5	0.8	1.3	1.6	31	3.5

Type B Expansion Shell

Number of tests	Head Plate		Installed load torque (lb–ft)	Initial tension (tonnes)	Loss after 1 day (tonnes)	Loss after 2 days (tonnes)	Loss after 7 days (tonnes)	Loss after 15 days (tonnes)	Loss after 32 days (tonnes)	Loss after 104 days (tonnes)	Total loss (%)	Residual tension (tonnes)
4	▫		169	4.5	0.3	0.3	0.6	0.8	1.2	1.2	27	3.3
4	▣		151	3.1	0.7	0.9	1.0	1.0	1.3	1.4	45	1.7
4	▦		166	2.9	0.5	0.6	0.6	0.8	1.2	1.3	45	1.6
4	▫	HSW	173	5.4	0	0.3	0.3	0.4	0.7	0.7	13	4.7
4	▣	HSW	154	4.9	0.4	0.6	0.6	1.0	1.4	1.9	39	3.0
4	▦	HSW	173	4.6	0.2	0.5	0.7	0.9	1.3	1.6	35	3.0

Fig. A.2 – Results of tension loss tests with flat support plates. From Potvin (A.6)

The following conclusions may be derived from an examination of the table:

- The tension losses seem to be independent of the type of anchor used.
- Most of the tension losses occur during the first seven days and generally stabilize after 32 days.
- The tension losses obtained when a plywood washer was added were not significantly higher than when the support plate was used alone. On the other hand, the loss of tension continued to increase slightly even after 32 days with the plywood washer.
- The addition of the HSW did not increase the loss of tension. On the other hand, since HSWs increase the initial tension in the bolts (a very significant increase for bolts type B expansion shell), the residual tension in the bolts after 104 days was considerably higher (from 3.0 to 4.7 tonnes instead of 1.6 to 3.8 tonnes).
- In general, we could use the table below as a guide for the use of 102 x 102 x 6.4 mm (4" x 4" x 1/4") head plates driven in as tight as possible with a stoper hammer (with a torque of 236 N-m (175 lb-ft) or more).

	Initial tension (tonnes)	Residual tension after 3 months (tonnes)
Plate alone	4.9	3.6
Plate with HSW	5.5	4.7
Plate with plywood washer on wire mesh	3.5	2.0
Plate with plywood washer on wire mesh with HSW	4.9	3.3

A.7 – RELATIONSHIP BETWEEN INSTALLED LOAD TORQUE AND TENSION IN EXPANSION BOLTS WITH AND WITHOUT A HARDENED STEEL WASHER

Three series of tests are discussed:

a) Stelco Tests (A.7)

Tests going back to 1965 were carried out with forged head (FH) bolts with diameters of 15.9 mm (5/8") and 19 mm (3/4"). The tension in the bolts was determined after the application of installed load torques of 229 N–m (170 lb–ft) and 330 N–m (245 lb–ft) respectively, in the following three cases:

A: bolt with hardened steel washer (HSW), threads clean and lubricated;

B: bolt with hardened steel washer, accumulation of small amounts of dirt and corrosion on the threads, no lubrication;

C: bolt with soft steel washer, accumulation of dirt and corrosion on the threads, no lubrication.

The results obtained are shown on the table below.

	Tension (tonnes)	Percentage of gain from bolt A
Bolt A diam. 15.9 mm (5/8")	3.0	–
Bolt B diam. 15.9 mm (5/8")	3.5	17
Bolt C diam. 15.9 mm (5/8")	4.9	64
Bolt A diam. 19 mm (3/4")	4.4	–
Bolt B diam. 19 mm (3/4")	5.2	18
Bolt C diam. 19 mm (3/4")	7.3	64

The improvement obtained when we go from situation A to situation C is about 64%, which is considerable. However, this can be attributed both to the lubrication of the threads and the use of an HSW. Since the threads are lubricated when they leave the plant, it would be interesting to determine the improvement solely caused by the use of the HSW. Upon an initial approximation, we could say that this corresponds to the difference between situations B and C; that is 40% for both the 15.9 mm (5/8") and 19 mm (3/4") bolts.

b) Inco Tests (A.6)

A series of torquing tests was carried out in an underground gallery with forged head (FH) bolts and double threaded (DT) bolts with and without HSW's. The tests consisted of installing the bolts and tightening them by successive increments with a stoper hammer. During each increment, the installed load torque was measured with a torque wrench, and the tension in the bolt was measured using a Gloetzl load cell placed between the wall of the gallery and the head plate.

The combinations tested are listed below. For each test, a graph was traced to express the relationship between tension in the bolt and installed load torque (see figures A.3 to A.6).

1) DT bolt, 102 x 102 x 6.4 mm (4" x 4" x 1/4") plate (graph 1);

2) DT bolt, 102 x 102 x 6.4 mm (4" x 4" x 1/4") plate, 152 x 152 x 12.7 mm (6" x 6" x 1/2") plywood washer (graph 2);

3) DT bolt, 102 x 102 x 6.4 mm (4" x 4" x 1/4") plate, 152 x 152 x 12.7 mm (6" x 6" x 1/2") plywood washer, welded no. 9 wire mesh (graph 3);

4) identical to 1, with an HSW with an internal diameter of 17.5 mm (11/16") (graph 4);

5) identical to 2, with an HSW with an internal diameter of 17.5 mm (11/16") (graph 5);

6) identical to 3, with an HSW with an internal diameter of 17.5 mm (11/16") (graph 6);

7) FH bolt, 102 x 102 x 6.4 mm (4" x 4" x 1/4") plate (graph 9);

8) FH bolt, 102 x 102 x 6.4 mm (4" x 4" x 1/4") plate, 152 x 152 x 12.7 mm (6" x 6" x 1/2") plywood washer (graph 10);

9) FH bolt, 102 x 102 x 6.4 mm (4" x 4" x 1/4") plate, 152 x 152 x 12.7 mm (6" x 6" x 1/2" plywood washer, welded no. 9 wire mesh (graph 11);

10) identical to 7, with an HSW with an internal diameter of 17.5 mm (11/16") (graph 13);

11) identical to 8, with an HSW with an internal diameter of 17.5 mm (11/16") (graph 14);

12) identical to 9, with an HSW with an internal diameter of 17.5 mm (11/16") (graph 15).

Finally, the results of tests 1 to 3, 4 to 6, 9 to 11, and 13 to 15 were combined in graphs 7, 8, 12, and 16 respectively. Thus, each of these four graphs represents the following conditions:

Graph 7: DT bolts without HSW
Graph 8: DT bolts with HSW
Graph 12: FH bolts without HSW
Graph 16: FH bolts with HSW

The following conclusions may be derived from the graphs:

- In spite of the wide scatter of points in all graphs, the trend seems to be a decrease in the tension of bolts installed with a plywood washer or a plywood washer with wire mesh. However, this decrease was significant only for bolts used without an HSW, as shown in the table below. Consequently, an HSW should be added to the FH and DT bolts when they are used to install the wire mesh.

- The table confirms the considerable increase in bolt tension obtained with the use of an HSW. Also, it would appear that the improvement is greater for DT bolts than for FH bolts. When FH bolts are used only in combination with a head plate and without a plywood washer and wire mesh, they already show a tension of 3.6 tonnes with a torque of 229 N-m (170 lb-ft).

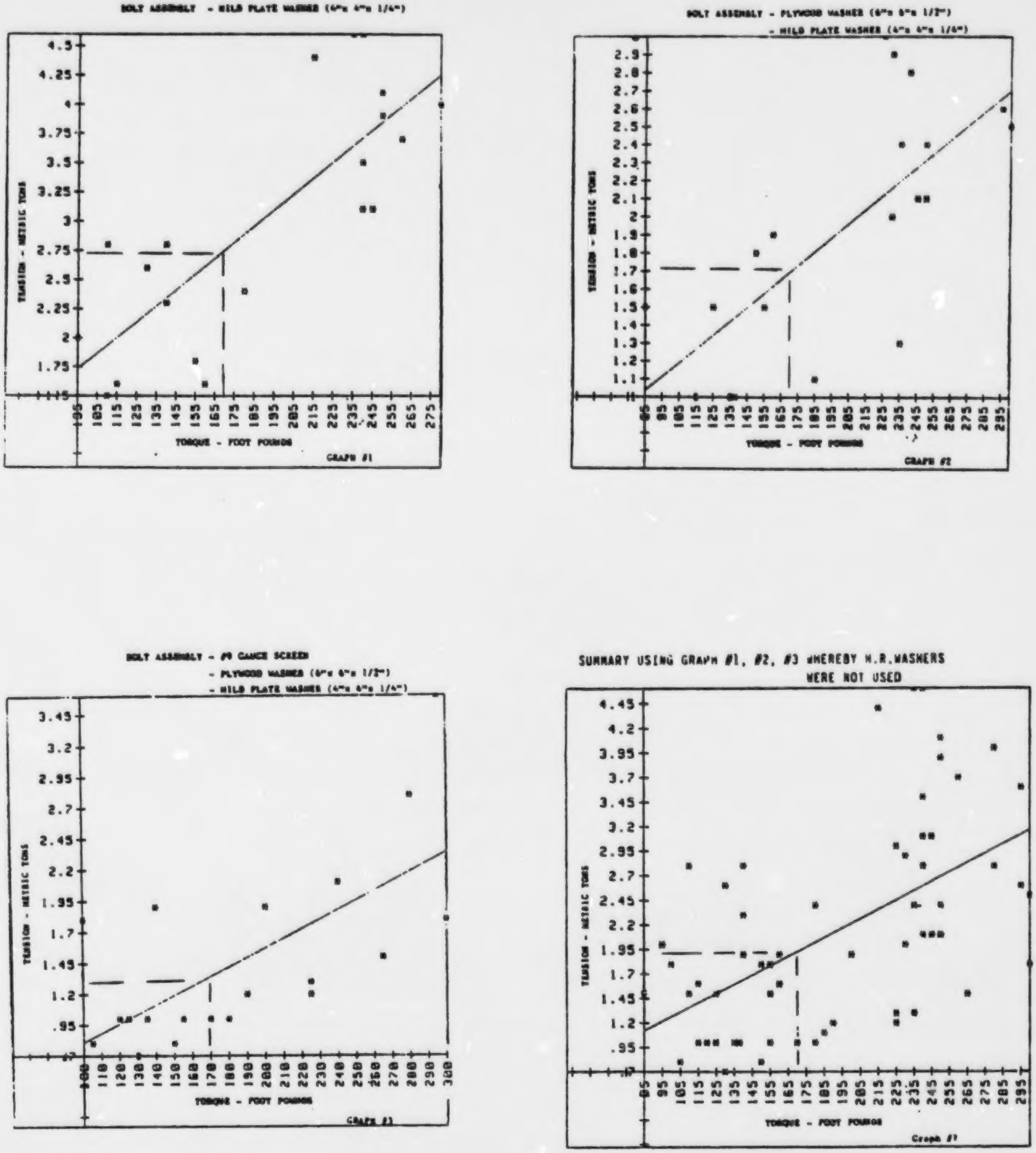
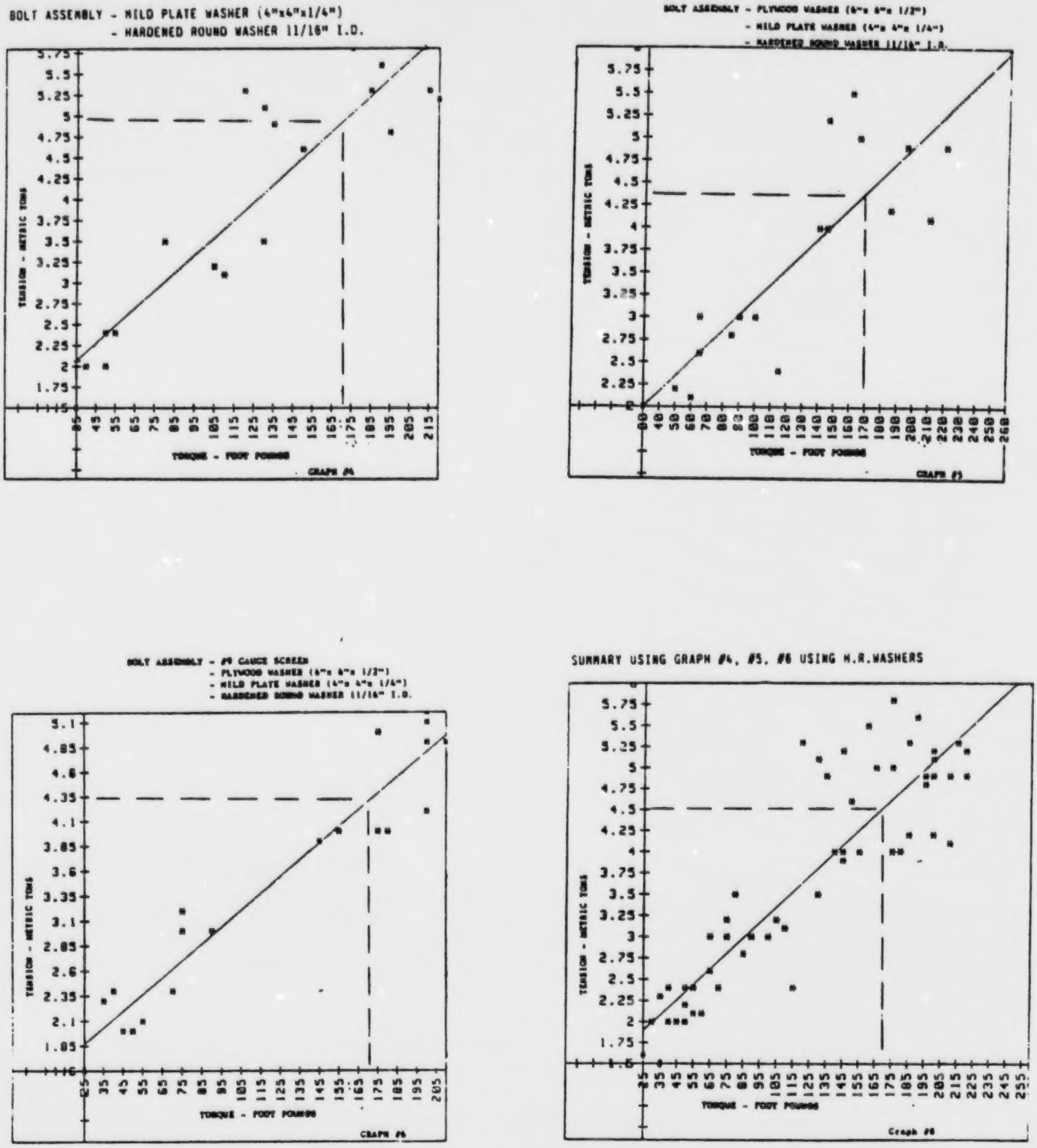


Fig. A.3 – Relationship between tension and installed load torque for DT bolts without an HSW. From Potvin (A.6)



BOLT ASSEMBLY - MILD PLATE WASHER (4"x4"x1/4")
- HARDENED ROUND WASHER 11/16" I.D.

BOLT ASSEMBLY - PLYWOOD WASHER (6"x 6"x 1/2")
- MILD PLATE WASHER (4"x 4"x 1/4")
- HARDENED ROUND WASHER 11/16" I.D.

BOLT ASSEMBLY - #9 GAUGE SCREEN
- PLYWOOD WASHER (6"x 6"x 1/2")
- MILD PLATE WASHER (4"x 4"x 1/4")
- HARDENED ROUND WASHER 11/16" I.D.

SUMMARY USING GRAPH #4, #5, #6 USING H.R.WASHERS

Fig. A.4 – Relationship between tension and installed load torque for DT bolts with an HSW. From Potvin (A.6)

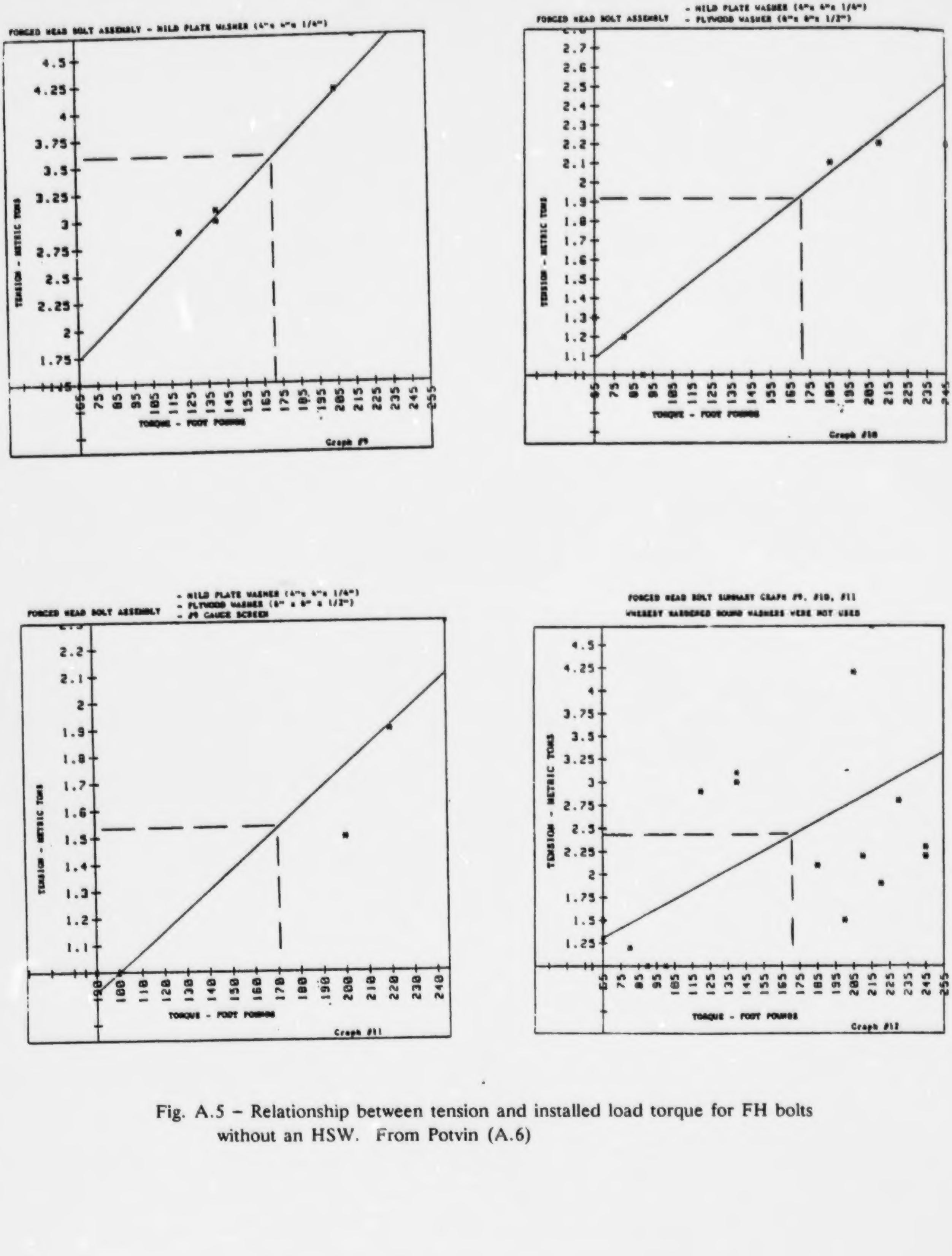


Fig. A.5 – Relationship between tension and installed load torque for FH bolts without an HSW. From Potvin (A.6)

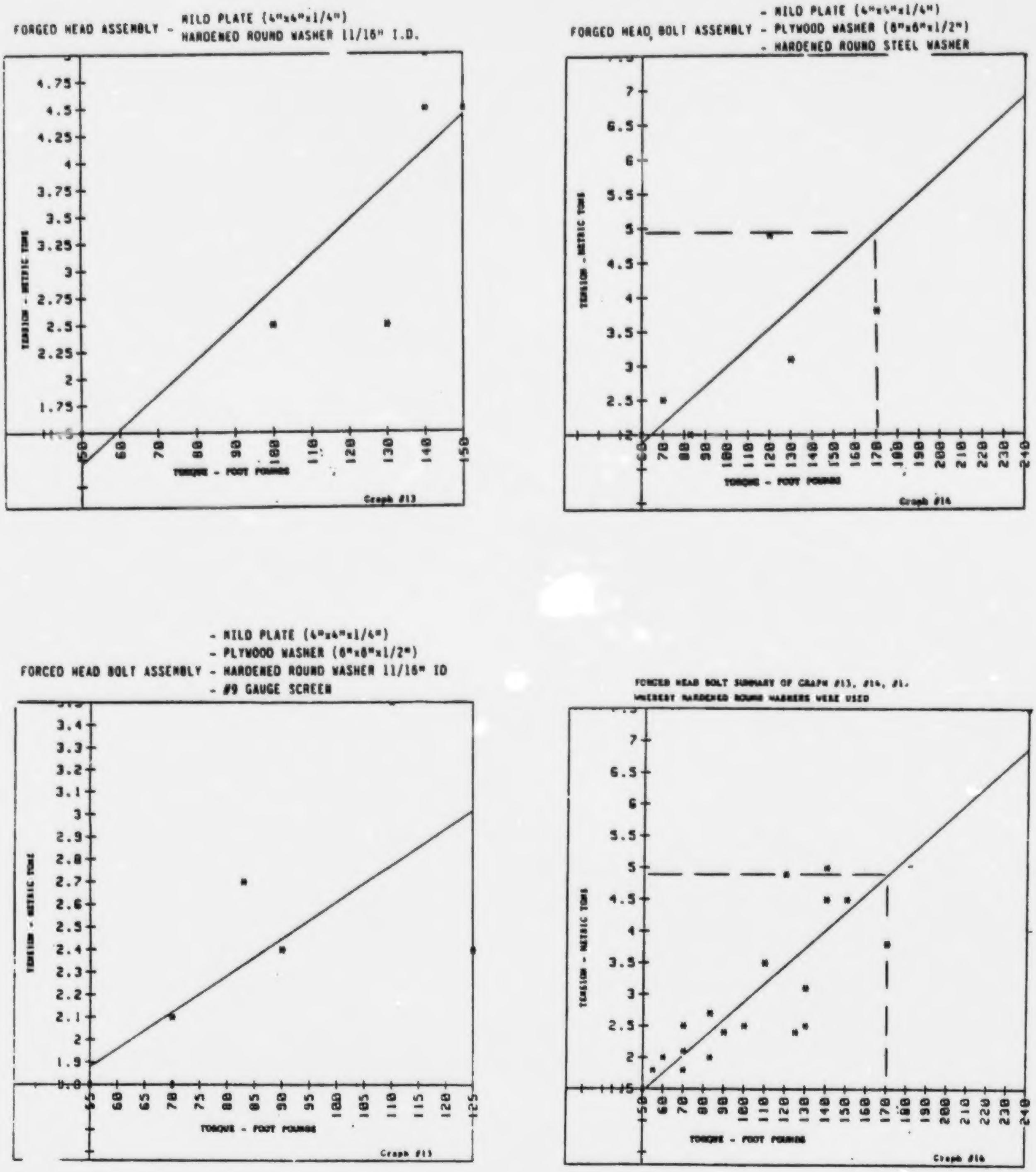


Fig. A.6 – Relationship between tension and installed load torque for FH bolts with an HSW. From Potvin (A.6)

However, we should point out that very few tests were carried out to arrive at the results shown in graph 9, and that additional tests would be necessary to confirm them. The USBM tests discussed below have nevertheless arrived at similar conclusions.

	Tension without HSW (torque: 170 lb–ft) (tonnes)	Tension with HSW (torque: 170 lb–ft) (tonnes)
DT bolt and plate	2.8	5
DT bolt, plate, plywood washer, and wire mesh	1.3	4.4
FH bolt and plate	3.6	5.1 (few measures)
FH bolt, plate, plywood washer and wire mesh	1.6	No measures at 170 lb–ft Extrapolated value: 3.4

- The author of the test also mentions an additional reason not to use an HSW with FH bolts installed using only a head plate: he observed shearing in the head when the bolts were installed at an angle to the wall. This shearing occurs with high installed load torques when the rotating bolt barrel buckles near the head. The same phenomenon did not take place with DT bolts installed at an angle, because the nut at the head turns independently of the bolt barrel.

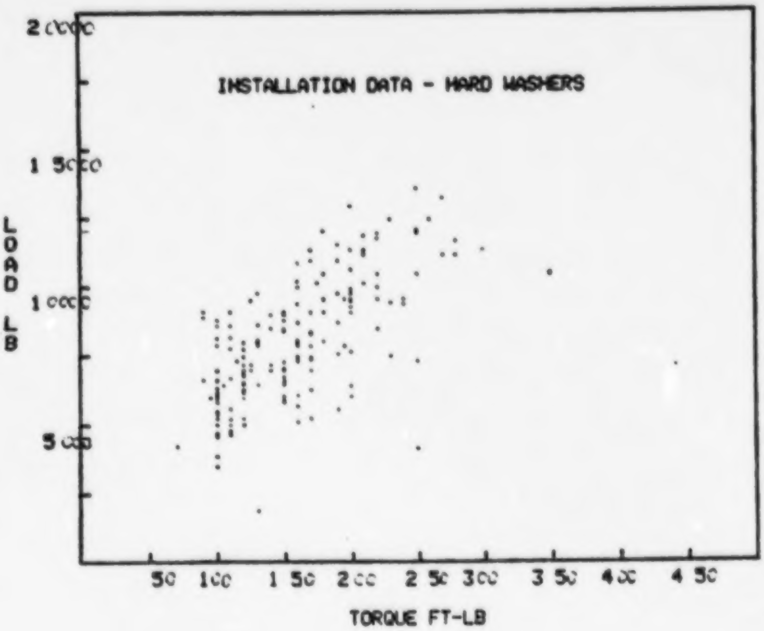
- In summary, the advantages of using an HSW in the circumstances described above are an increase in the tension in the bolts, better consistency in the relationship between installed load torque and tension in the bolt (compare graphs 7 and 9, and 12 and 16), the possibility of obtaining a sufficiently high level of tension in the bolt itself when the installed load torque is only 162 N–m (120 lb–ft), and, finally, greater ease of installation and less time needed during installation of the bolt to tighten it to the level of tension required.

c) USBM Tests (A.8)

As part of a comprehensive measuring campaign carried out in an underground coal mine in Utah, initial tension and installed load torque were measured in 400 bolts equipped with a load cell. Half of the bolts were installed with HSWs. The bolts had forged heads and were installed with a head plate. The enclosing rock in the roof of the galleries was sandstone.

Figure A.7 shows the graph illustrating initial tension measured as a function of installed load torque in both cases (with and without HSW), and Figure A.8 shows the histograms of tensions measured for installed torque values between 202 and 230 N–m (150 and 170 lb–ft).

The conclusion derived from these tests is that the general relationship between installed load torque and tension in the bolts shows the same scatter with or without the use of HSWs for FH bolts. When the applied load torque is relatively uniform (202 and 230 N–m (150 to 170 lb–ft)), the initial tension in the bolts is also more uniform, both with an HSW (2.3 to 4.3 tonnes (5,000 to 9,500 lb)) and without an HSW (1.1 to 5.4 tonnes (2,500 to 12,000 lb)).



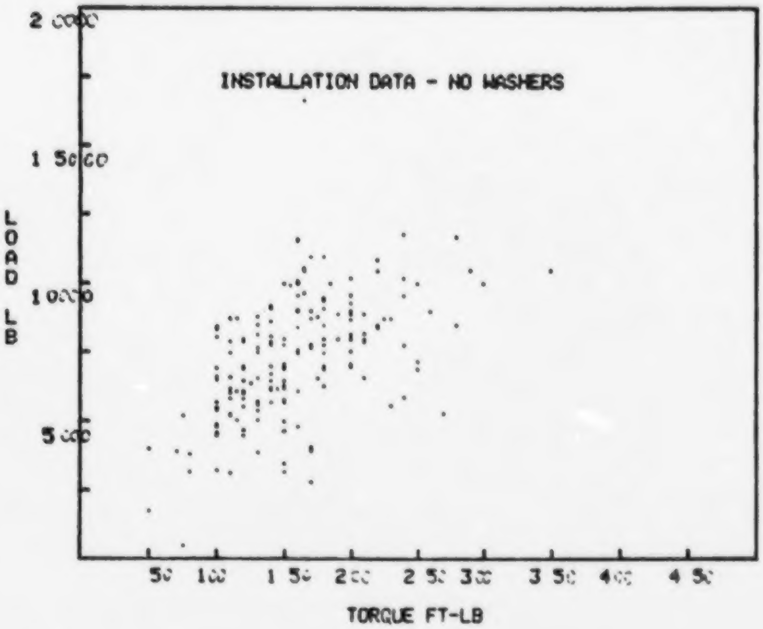


Fig. A.7 – Relationship between tension and installed load torque for FH bolts with and without an HSW. From Rosso (A.8)

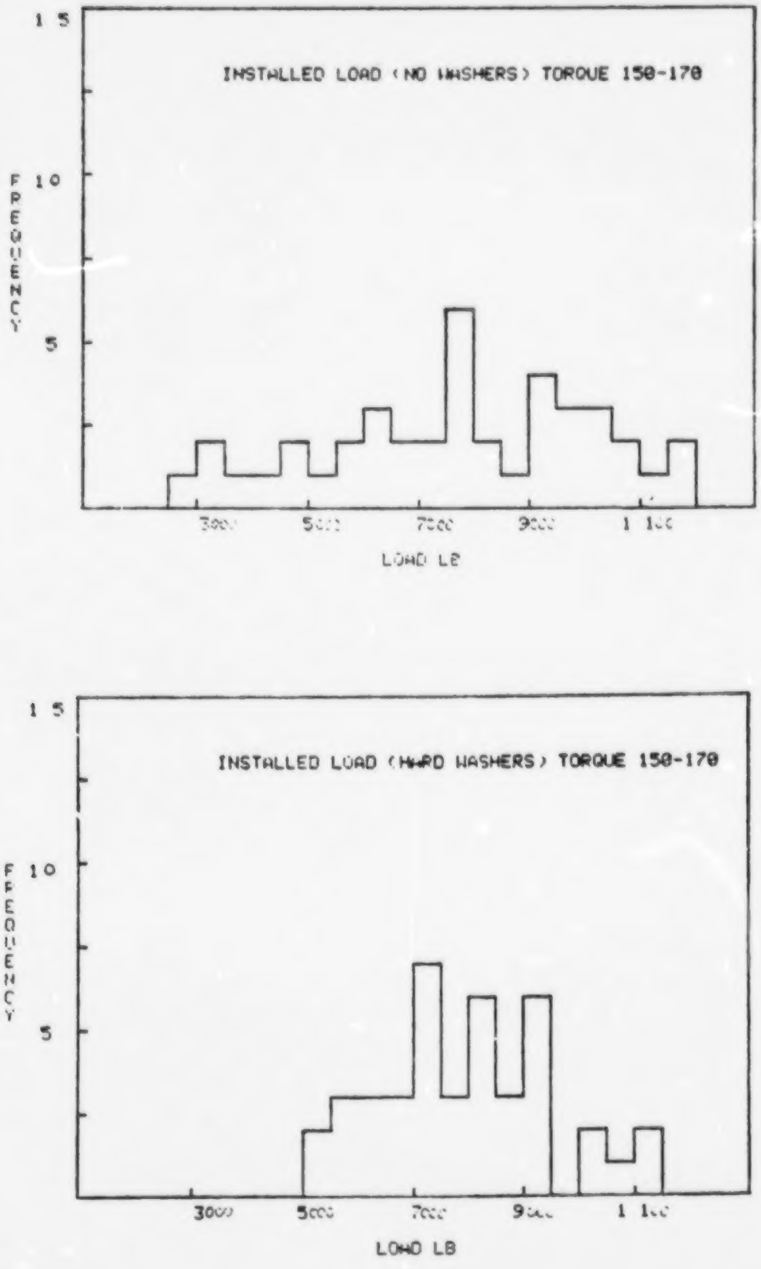


Fig. A.8 – Tension histograms measured for an installed load torque between 202 and 230 N–m (150 and 170 lb–ft), with and without an HSW. From Rosso (A.8)

A.8 – LOADING TESTS ON VARIOUS TYPES OF WIRE MESH

The tests discussed are those of the Ministry of Labour of Ontario (A.9). They were carried out in an underground gallery with the experimental device shown in Figure A.9. The wire mesh is kept in place by four rockbolts separated by a space of 1.2 m (4 ft), and is loaded in the centre using a 0.3 x 0.3 m (1 ft x 1 ft) steel pulling plate connected to a chain block. The bolts used during the tests were double threaded and were installed with a 102 x 102 mm (4" x 4") head plate and a 152 x 152 mm (6" x 6") plywood washer.

The table in Figure A.10 summarizes the types of wire mesh tested, as well as the maximum load obtained and the corresponding displacement in the centre of the mesh.

The following conclusions may be derived from the tests.

- All the tests caused the gradual rupture of the wire mesh, with a few wires giving way one after the other until the maximum load was reached. Beyond the maximum load, there was often a relatively sudden drop in the load-bearing capacity, although the mesh was not completely pulled out and there were no other extreme phenomena.

- Welded wire meshes have a displacement capacity of about 280 mm (11") regardless of the thickness of the wires, while chain link meshes have a higher capacity that may reach 406 mm (16").

- The maximum load reached by the different meshes was relatively high 13.9 to 36.0 kN: (3,100 lb to 8,100 lb) and seemed to be proportional to the thickness of the wires, particularly in the case of the welded wire meshes.

- During loading, the first wires to give way are those that are in contact with the anchoring bolts. This may mean that the plywood washers play an important role in determining the results of the tests, and that the performances observed would have been different with anchoring bolts and head plates without plywood washers.

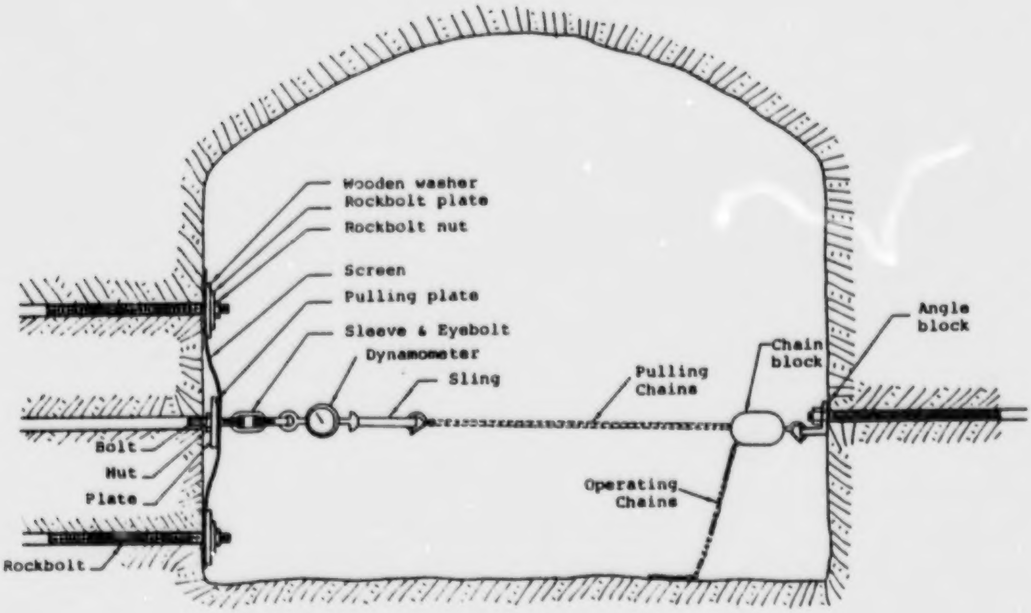

Wooden washer
Rockbolt plate
Rockbolt nut
Screen
Pulling plate
Sleeve & Eyebolt
Dynamometer
Sling
Pulling Chains
Chain block
Angle block
Bolt
Nut
Plate
Operating Chains
Rockbolt

Fig. A.9 – Experimental device used to load the wire mesh in an underground gallery. From Pakalnis *et al.* (A.9)

Wire mesh	Maximum load (lb)	Displacement in the centre of the mesh (in)
4" x 4" no. 4 gauge welded mesh	8,100	11.5
4" x 4" no. 6 gauge welded mesh	7,400	11.25
4" x 4" no. 9 gauge welded mesh	4,200	10.75
4" x 2" no. 12 gauge welded mesh	3,100	12.25
2" x 2" no. 9 gauge chain link mesh (average of 3 tests)	6,300	16.5
2" x 2" no. 11 gauge chain link mesh (average of 2 tests)	5,200	15

Fig. A.10 – Results of wire mesh pullout tests. From Pakalnis *et al.* (A.9)

A.9 - TIGHTENING OF DOUBLE-THREADED EXPANSION BOLTS

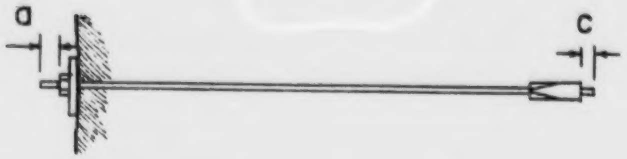
The tests discussed were carried out by Inco (A.6). They consisted of installing bolts in an underground gallery and measuring the free length of the bolt barrel at the two ends (the threaded wedge and the nut protruding outside) before and after driving in the bolt as tight as possible with the stoper hammer (see Fig. A.11). To measure the end inside the drill hole, the tension of the bolt had to be released first, using a torque roof bolt tensioner. Two types of expansion shells were used during these tests.

The table shown in Figure A.11 summarizes the results of these tests, from which we can derive the following conclusions:

- When the DT bolt is tightened, both the protruding nut and the threaded wedge of the expansion shells advance by the same distance 23 mm (0.9"). The distance travelled is the same for the two types of expansion shells tested.
- This result is valid only when the bolt is installed perpendicular to the wall. When it is installed at an angle, the bolt barrel tends to bend and this impedes rotation. Thus, most of the distance travelled will be on the protruding nut.

Test	Type of expansion shell	Before tightening		After tightening	
		A (in)	B (in)	A (in)	B (in)
1	B	1	0	1.75	0.75
2	A	0.875	0	1.375	0.875
3	B	1	0	2	0.625
4	A	0.875	0	1.75	0.625
5	A	1	0	2.375	1.063
6	B	1	0	1.625	0.875
7	A	1	0	1.94	1.063
8	A	1	0	2.125	1.125
9	B	1	0	2	0.875
10	B	1	0	1.875	1.25
Mean (shell A)		0.95	0	1.91	0.95
Mean (shell B)		1	0	1.85	0.88

Before tightening:



After tightening:

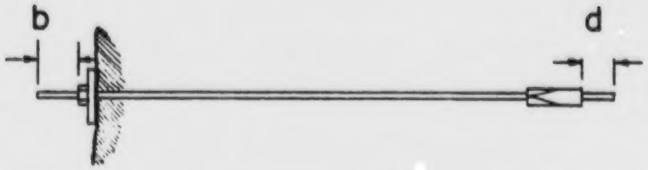


Fig. A.11 – Results of tightening tests on DT bolts. From Potvin (A.6)

A.10 –TORQUE LOSS IN MECHANICALLY ANCHORED BOLTS AFTER DRIFT BLASTING

The results shown are from a test carried out in 1983 by an underground mine in Quebec. The name of the mine and company is confidential. The purpose of the test was to determine how much bolt tension was lost because of blasting vibrations. Five drift blasts were studied. The bolts were installed right behind the working face, and the initial installed load torque was obtained. After the blast, the torque was read again, the bolts were retightened with a torque wrench, and readings were obtained at regular intervals for a few days.

The results are shown in the table in Figure A.12, from which the following conclusions may be derived:

- Tension losses because of blasting vibrations were very low for these bolts; they went from an average torque of 248 N–m (184 lb–ft) to a torque of 236 N–m (175 lb–ft).
- The low tension loss may be attributable to the high initial installed load torque.
- Tightening with a torque wrench makes it possible to increase bolt torque significantly.

Blast	Number of bolts	Installed load torque (lb–ft)	Torque after blasting (lb–ft)	Torque after retightening	Torque at 1st reading (lb–ft) (days elapsed)	Torque at 2nd reading (lb–ft) (days elapsed)	Torque at 3rd reading (lb–ft) (days elapsed)	Torque at 4th reading (lb–ft) (days elapsed)
1	12	166	170	246	228 (5)	232 (6)	227 (8)	233 (12)
2	9	195	174	231	211 (1)	233 (3)	232 (7)	–
3	6	170	174	271	244 (2)	237 (6)	–	–
4	5	196	185	257	240 (2)	–	–	–
5	10	195	178	224	–	–	–	–
Mean		184	175	245	–	–	–	–
Standard deviation		30	30	30	–	–	–	–

Figure A.12 – Installed load torque measures obtained before and after drift blasting in an underground mine

A.11 - RESULTS OF PULL-OUT TESTS TO VERIFY THE ANCHORING QUALITY OF MECHANICALLY ANCHORED BOLTS

Three series of tests are discussed.

a) Inco Tests (A.6)

The purpose of the tests was to determine the failure load or pull-out load of mechanically anchored bolts equipped with five different types of expansion shells and installed in drill holes with nominal diameters of 31.75 mm (1 1/4") and 34.9 mm (1 3/8"). Among shells tested, A, B, and E were designed for use with 1 1/4" holes (the acceptable tolerance in the diameter of the hole recommended by the manufacturers is +1.6 mm (+1/16") and -0.8 mm (-1/32"); shells C and D can be used indiscriminately in holes with diameters of 1 1/4" to 1 3/8". A photograph of the expansion shells tested is shown in Figure A.13.

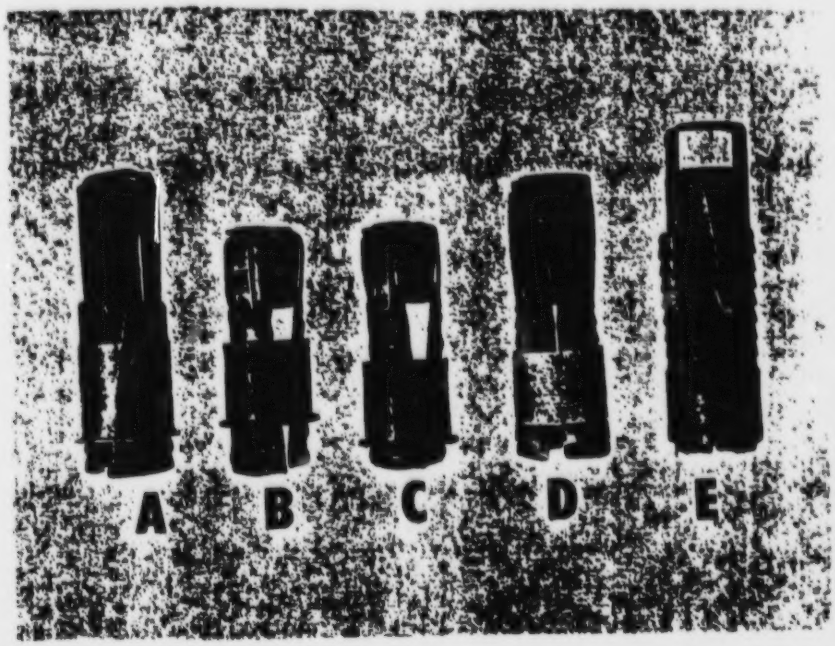


Fig. A.13 - Expansion shells for 1 1/4" and 1 3/8" drill holes. From Potvin (A.6)

The tests were carried out using a pull-out device that included a 60 tonne jack with a 152 mm (6") stroke. The diameter of the holes was measured using a diameter gauge (see Fig. A.15). The tests were carried out in two types of grounds: sulphide rock masses

(Creighton Mine), and waste rocks (Stobie Mine). The results of the tests are shown in the table in Figure A.14, from which the following conclusions may be derived:

- Even though the load at rupture of the 15.9 mm (5/8") bolt barrels guaranteed by the manufacturers was only 100 kN (22,600 lb) (Section 3.1.1), in practice, much higher values, about 124 kN (28,000 lb) were obtained.
- In several tests, there was premature slip of shells A and B in the sulphide rock masses. Thus, anchoring in this type of ground should be more systematically studied.
- In the holes with diameters of 35.8 mm (1.41") to 36.3 mm (1.43") (about 4.7 mm (3/16") above the nominal diameter of 31.75 mm (1 1/4")), there was a very significant drop in the anchoring quality in the case of the A, B, and E anchors (from 14 tonnes to about 5 to 8 tonnes).
- In the same holes, shells C and D retained all their anchoring quality.

Mine	Diameter of the bit (in)	Diameter of the hole (in)	Expansion shell	Ground	Type of rupture	Maximum load reached (tonne)	Comments
Creighton	1.25	1.28	A	sulphide	AS	11	poor anchoring
		1.29	A	sulphide	AS	11	poor anchoring
		1.26	B	sulphide	BR	14	good bolt
		1.29	B	sulphide	AS	11	poor anchoring
		1.28	C	sulphide	BR	14	good bolt
		1.30	C	sulphide	BR	14	good bolt
Stobie	1.25	1.28	A	rock	BR	14	good bolt
		1.29	B	rock	BR	13.5	good bolt
		1.30	C	rock	BR	13.5	good bolt
	1.375	1.41	A	rock	AS	8	poor anchoring
		1.43	A	rock	AS	8	poor anchoring
		1.41	A	rock	AS	4.5	poor anchoring
		1.41	B	rock	AS	8	poor anchoring
		1.41	B	rock	AS	5.5	poor anchoring
		1.43	B	rock	AS	5.5	poor anchoring
		1.42	C	rock	BR	12.4	good bolt
		1.43	C	rock	BR	14	good bolt
		1.42	C	rock	BR	13	good bolt
		1.42	D	rock	BR	14	good bolt
		1.42	D	rock	BR	15	good bolt
		1.43	D	rock	BR	14	good bolt
		1.41	D	rock	BR	13	good bolt
		1.41	E	rock	AS	7	poor anchoring
		1.42	E	rock	AS	9.5	poor anchoring
		1.41	E	rock	AS	8	poor anchoring
		1.42	E	rock	AS	7.5	poor anchoring
		1.41	E	rock	AS	7	poor anchoring

AS: Anchor slip
BR: Bolt rupture

Fig. A.14 – Results of pull-out tests on five different types of anchors. From Potvin (A.6)

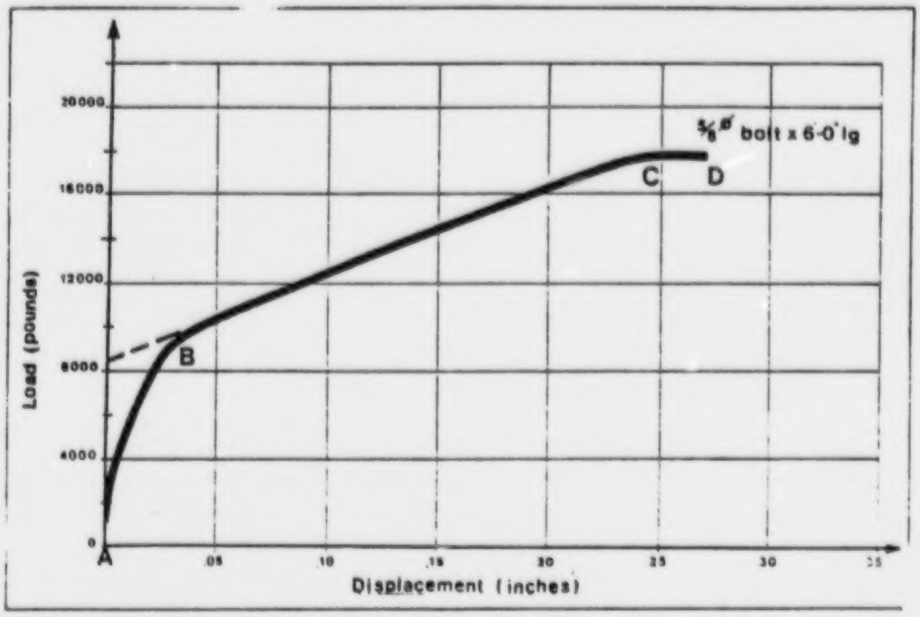


Fig. A.15 – Loading curve during rockbolt pullout tests. From Ames *et al.* (A.10)

b) Tests Carried Out by the Ontario Ministry of Labour (A.10)

During the last few years, the technical group of the Mining Health and Safety Branch, located in Sudbury, has placed great emphasis on carrying out field tests to verify the anchoring quality of rockbolts and the installation procedures used. We should point out that this effort has gone hand in hand with the introduction of the following section in the Ontario regulations (Regulations for Mines and Mining Plants, Revised Regulations of Ontario, Regulation 694, Section 68):

> Rockbolts used to secure the enclosing rock in an underground mine shall be properly installed and a proportion thereof shall be pulltested and, in the case of torque–tension bolts, be torque tested for proper installation and adequacy of materials used.

Since 1978, a total of 250 pull–out tests have been carried out on bolts under many tightening conditions (low and high installed load torque) installed in drill holes of different diameters.

To judge the anchoring quality of bolts the ministry chose an anchor slip criterion initially proposed by Underwood and Distefano (A.11) for NORAD permanent excavations. This criterion is evaluated on the bolt loading curve shown in Figure A.15, and states that the anchor slip must not exceed 1.6 mm per tonne (0.028" per 1,000 lb) of tension.

In Figure A.15, there are three sections. Section AB corresponds to the load applied to compensate for the initial tension caused by the tightening of the bolt. Section BC includes the elastic elongation of the bolt barrel and anchor slip (as well as the portion of the displacement that corresponds to its tightening). In this section, BC anchor slip is calculated by subtracting the portion caused by elastic elongation (which must be evaluated separately using a laboratory loading test). Point B, that is, its projection to the axis of the ordinate, is considered to be the initial tension in the bolt, and point C is the yield strength. Section CD corresponds to the progression of anchor slip and the elongation of the bolt barrel to rupture. This section of the curve was generally not recorded completely, because of the risks involved if a sudden rupture occurred.

Of the conclusions derived from these tests, we emphasize the following:

- When the bolts were installed in drill holes that complied with the diameter tolerances recommended by the manufacturers, the geological nature of the ground had very little effect on anchor slip (Figure A.16). Moreover, the slip observed was much less than the allowable limit of 1.6 mm per tonne (0.028" per 1,000 lb).

- Similarily, when the drill hole was within the recommended tolerances, the five types of expansion shells found in the mines visited complied with the allowable slip limit (Figure A.17).

- The yield strength of the steel used to make the bolt barrels (point C) was generally reached when the load was 75.5 to 84.4kN (17,000 to 19,000 lb) while the yield strength guaranteed by the manufacturers is 60.2 kN (13,560 lb).

- Even though the installed load torque recommended by the manufacturers is 229 N-m (170 lb-ft), the bolts are often installed using no more than 162 N-m (120 lb-ft). In spite of this, many test results with bolts tightened using these or lower values showed anchor slip near or under the allowable limit of 1.6 mm per tonne (0.028" per 1,000 lb); the initial tension was within the acceptable limit of 3.0 tonnes, and the load exceeding the yield strength was 9.4 tonnes, similar to that of bolts tightened with 229 N-m (170 lb-ft). The results of tests on bolts tightened with a low installed load torque are shown in Figure A.18. For purposes of comparison, the results of the tests carried out on bolts installed with torques of 215 to 242 N-m (160 to 180 lb-ft) and 270 to 323 N-m (200 to 240 lb-ft) are shown in Figures A.19 and A.20.

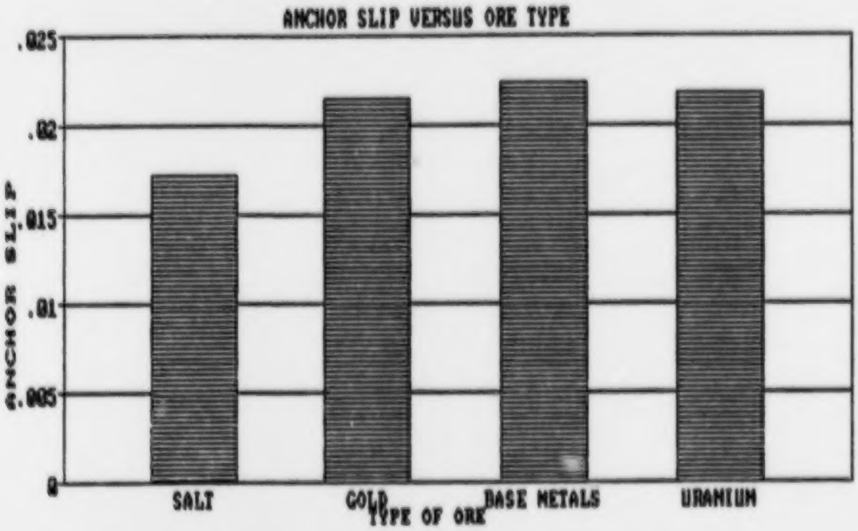


Fig. A.16 – Anchor slip in different types of ground. From Ames *et al.* (A.10)

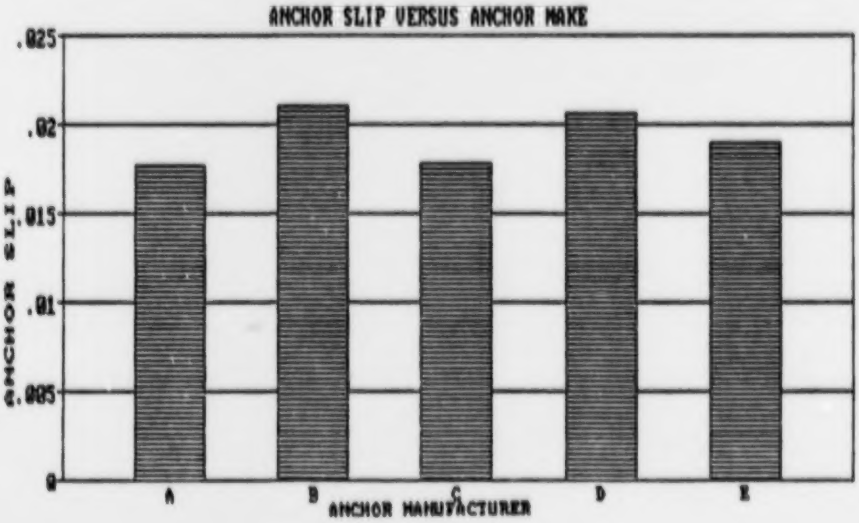


Fig. A.17 – Anchor slip in different types of mechanically anchored bolts. From Ames *et al.* (A.10)

Mine	Installed load torque (lb–ft)	Diameter of the drill hole (in)	Initial tension (tonnes)	Load exceeding the yield strength (tonnes)	Anchor slip (in/1,000 lb)
1	95	1.31	3.7	10.8	0.0305
2	120	1.32	3.0	9.5	-
	115	-	1.5	9.0	0.061
	115	-	2.6	9.5	0.0198
	110	-	3.0	10.0	0.0296
	110	-	4.1	9.0	0.0150
	105	-	3.2	9.0	0.0246
	120	-	3.3	9.5	0.0296
	115	-	4.5	9.5	0.0290
	110	-	2.3	9.5	0.0252
4	75	1.25	4.0	11.2	-
5	120	1.39	4.6	7.0	0.0400
	115	1.39	3.5	8.5	0.0450
	90	1.39	3.4	8.0	0.0340
	110	1.39	3.5	8.5	0.0220
	90	1.39	1.8	9.0	0.0080
	100	1.39	1.8	9.0	0.0280
8	70	1.40	4.0	10.0	-
	90	1.40	4.4	10.0	-
	95	1.42	3.0	9.6	-
	90	1.42	5.0	9.0	-
9	120	1.27	3.7	8.5	0.0285
	115	1.27	2.6	8.2	0.0169
	60	1.27	2.6	8.2	0.0727
	80	1.27	2.9	8.5	0.0250
10	85	1.27	3.9	8.5	0.0178
	65	1.27	2.9	8.5	0.0360
	65	1.27	2.7	8.6	0.0300
12	45	1.25	1.7	12.3	0.0920
13	70	1.25	2.5	8.5	0.0580
	75	1.25	3.8	8.5	0.0260
15	80	1.29	4.0	10.6	0.0309
	80	1.29	2.0	10.0	0.0267
	50	1.28	1.8	10.0	0.0353
	40	1.29	1.8	9.0	0.0327
18	120	1.28	2.6	9.0	0.0206
19	115	1.27	3.2	10.0	0.0260
	115	1.23	2.4	10.0	0.0093
21	100	1.21	2.2	10.2	-
23	115	1.21	1.5	8.8	-
24	75	1.28	2.2	10.0	0.0178
	90	1.28	3.2	9.3	0.0225
	100	1.27	3.0	9.5	0.0220
29	90	1.26	1.2	9.4	0.0255
	110	1.28	1.6	9.4	0.0241
30	95	1.31	4.0	13.0	0.0077
mean	93.8	1.30	3.0	9.4	0.0292
standard deviation	21.9	0.06	0.95	1.06	0.0160

Fig. A.18 – Results of pull-out tests on bolts installed with a torque of less than 162 N–m (120 lb–ft). From Ames (A.12)

Mine	Installed load torque (lb-ft)	Diameter of the drill hole (in)	Initial tension (tonnes)	Load exceeding the yield strength (tonnes)	Anchor slip (in/1,000 lb)
1	160	1.30	4.6	10.0	0.0324
	160	1.31	4.2	10.0	0.0330
2	180	1.31	6.0	9.5	-
	170	1.36	4.0	8.5	-
	170	1.34	4.0	9.0	0.0173
	180	1.32	3.6	9.0	0.0085
	165	-	4.8	9.0	0.0170
	175	-	4.9	9.5	0.0200
	175	-	3.8	9.0	0.0220
	165	-	4.7	9.0	0.0226
	170	-	4.0	9.0	0.0245
10	180	1.27	2.0	8.4	-
11	160	1.25	0.5	8.2	-
	170	1.27	2.2	8.2	-
12	175	1.25	3.5	12.6	0.0330
	175	1.25	5.2	12.5	0.0176
13	175	1.25	2.2	8.2	-
14	170	1.30	3.6	10.6	-
16	175	1.34	4.8	9.0	0.0288
	160	1.29	2.4	8.0	0.0313
	160	1.29	3.4	9.0	-
	185	1.30	5.0	9.0	0.0383
	160	1.31	7.4	9.2	-
17	180	1.31	3.0	9.0	0.0196
18	160	1.34	0.9	9.0	0.0346
	170	1.28	0.5	9.0	0.0366
20	160	1.25	1.6	6.4	-
21	170	1.21	3.6	7.6	-
	165	1.21	4.3	7.5	-
23	170	1.21	3.2	8.8	-
25	160	-	4.2	6.5	0.0600
	180	-	6.3	8.6	0.1290
	180	-	5.6	9.0	0.0321
	170	-	4.7	9.0	0.0195
	165	-	3.5	-	0.0197
	170	-	1.0	8.0	0.0113
	170	-	4.3	10.0	0.0194
	175	-	2.8	8.5	0.0180
26	180	1.34	2.0	9.0	0.0430
	175	-	3.0	8.8	0.0070
mean	170.4	1.29	3.6	9.0	0.0295
standard deviation	7.4	0.04	1.6	1.2	0.0229

Fig. A.19 – Results of pull-out tests on bolts installed with a torque of 215 to 242 N-m (160 to 180 lb-ft). From Ames (A.12)

Mine	Installed load torque (lb-ft)	Diameter of the drill hole (in)	Initial tension (tonnes)	Load exceeding the yield strength (tonnes)	Anchor slip (in/1,000 lb)
1	210	1.28	4.0	9.0	0.0126
	210	1.28	5.0	9.0	0.0142
2	200	-	3.0	8.5	-
	240	-	4.8	9.0	0.0106
	200	-	2.9	9.0	0.0100
	205	-	1.0	9.0	0.0113
10	235	1.25	1.6	8.4	-
11	205	1.27	0.5	8.2	-
12	220	1.25	5.2	12.3	0.0208
13	205	1.25	2.5	8.4	-
14	220	1.27	4.0	10.0	0.0407
16	200	1.33	2.2	9.0	0.0281
	200	1.29	4.5	8.5	-
19	200	1.24	2.8	9.0	0.0290
25	200	-	4.8	8.6	0.0810
	210	-	6.0	9.0	0.0920
	200	-	6.0	9.0	0.0109
	230	-	5.5	9.0	0.0141
	200	-	3.9	9.0	0.0570
	210	-	5.3	11.0	0.0102
	225	-	4.4	9.0	0.0190
26	220	1.34	0.5	9.5	0.0470
	220	1.34	4.0	9.0	0.0270
	230	1.38	4.5	9.0	0.0020
28	200	1.27	-	8.5	0.0120
	200	1.28	4.2	9.5	0.0370
30	225	1.31	5.0	12.0	0.0130
mean	211.9	1.29	3.8	9.2	0.0273
standard deviation	12.6	0.04	1.6	1.00	0.0237

Fig. A.20 – Results of pull-out tests on bolts installed with a torque of 270 to 323 N-m (200-240 lb-ft). From Ames (A.12)

- The means and standard deviations shown in these three figures are very similar, for the anchor slip (0.0273 to 0.0295"/1,000 lb), the load exceeding the yield strength of steel (9.0 to 9.2 tonnes), and the initial tension, which increased only slightly from 3.0 to 3.8 tonnes. It is important to point out that all the bolts tested were generally installed in drill holes with diameters that complied with the tolerances recommended by the manufacturers (average calculated diameters in the figures were 1.29" to 1.30").

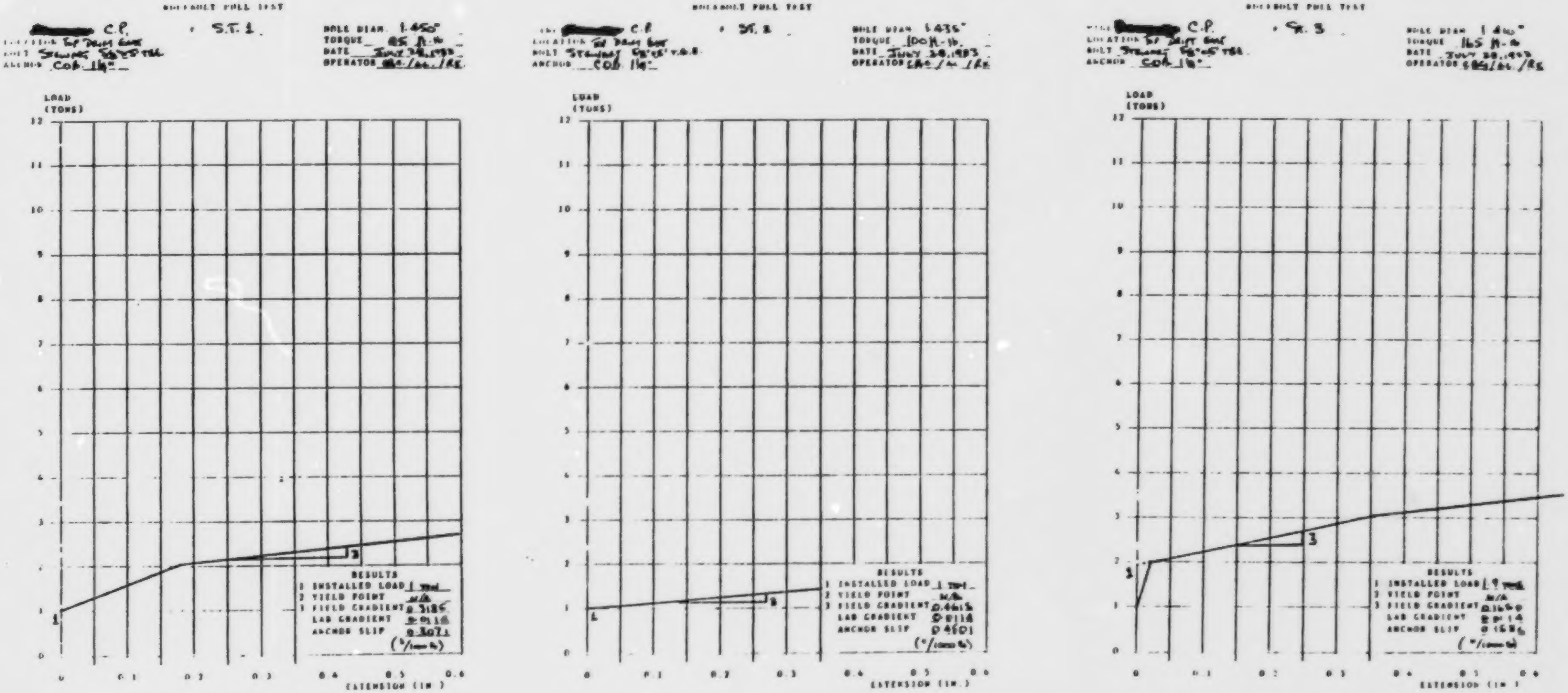
- These results confirm that the most important parameter that must be checked during the installation of the bolts is the diameter of the drill hole.

- The test curves shown in Figure A.21 are included as a specific example of the dominant effect of the diameter of the drill hole in relation to the installed load torque. These represent the results of nine pullout tests on 1.52 m (5 ft) DT mechanically anchored bolts installed in 1 1/4" holes. The tolerance of the diameter of the drill hole recommended by the manufacturer for the shells of the bolts is +1/16" and -1/32". The holes were drilled with 1 3/8" bits (tests 1 to 3), 1 1/4" bits (tests 4 to 6), and 1 7/32" bits (tests 7 to 9). The results of the tests are summarized in the table below.

- Tests 1 and 2 showed unacceptable performances, because the maximum load reached was under 3 tonnes with an anchor slip that was about ten times the limit of 1.6 mm per tonne (0.028" per 1,000 lb). The installed load torque of 222 N-m (165 lb-ft) in test 3 improved the situation only slightly and the gain was probably mainly caused by the decreased diameter of the hole (from 1.45" to 1.41").

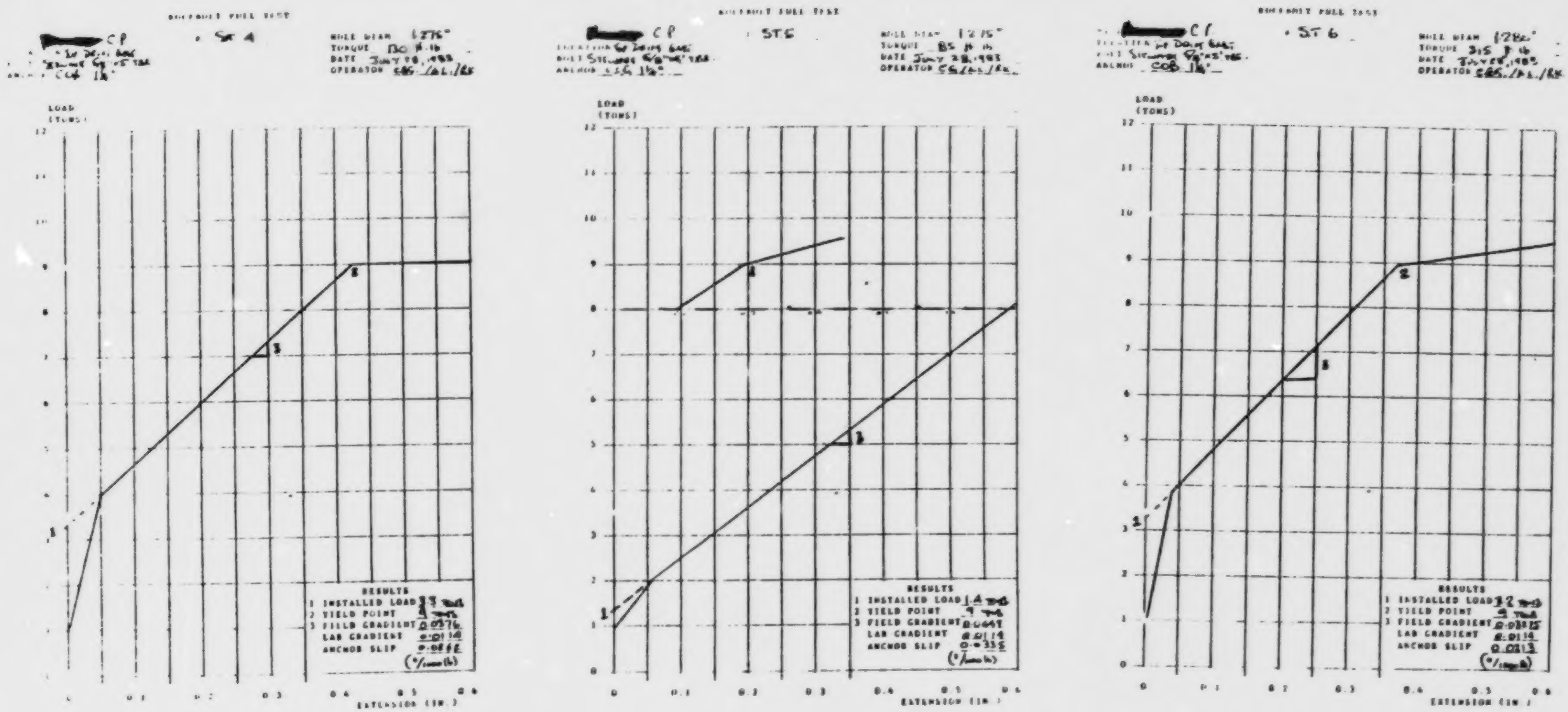
- Tests 4 to 6 produced satisfactory performances even though the results were slightly more than the allowable slip limit for test 5, with a very low installed load torque of 115 N-m (85 lb-ft).

- Tests 7 to 9 (1 7/32" bit) produced high slip values that probably reflect the improper installation of the anchor in a hole that was too close to or even less than the minimum allowable diameter tolerance.



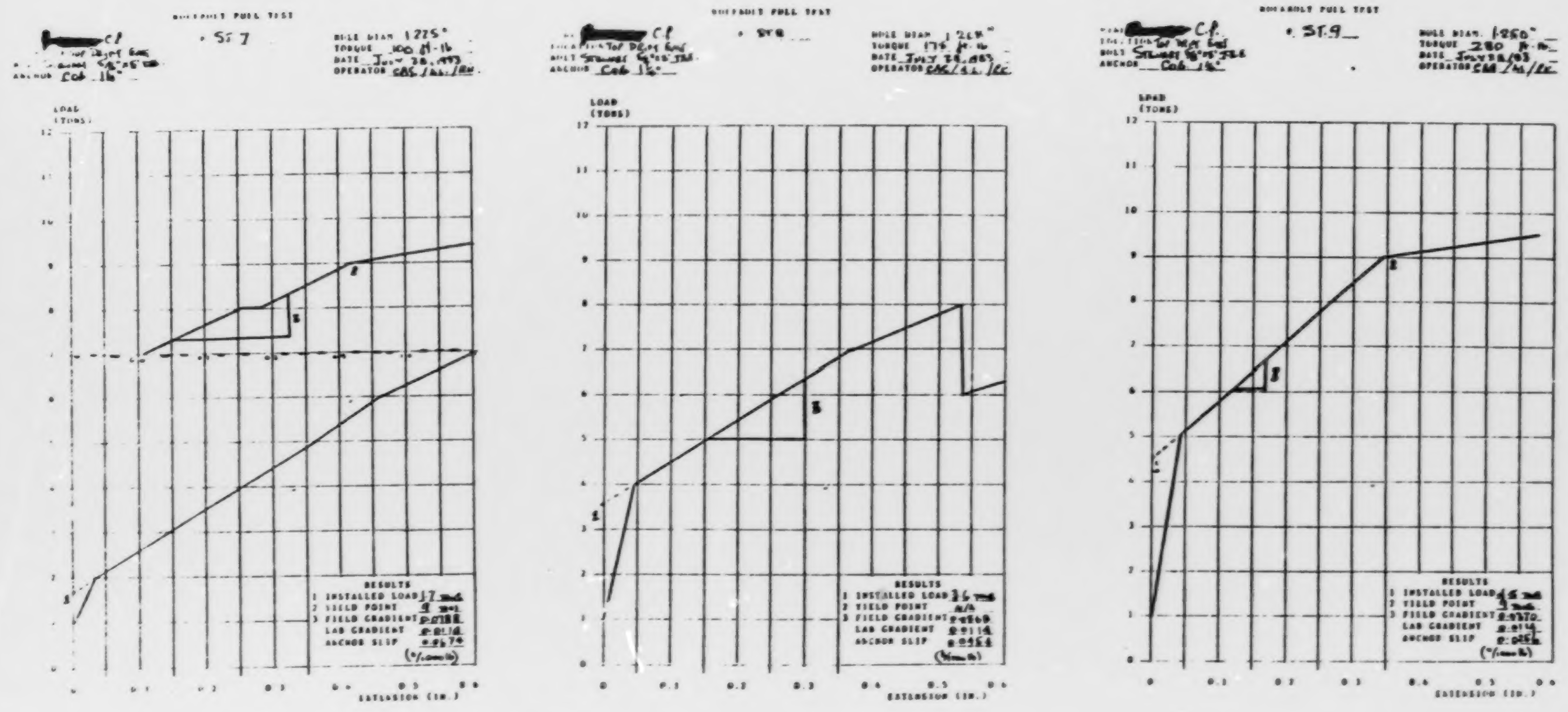
a) Tests in holes drilled with 34.9 mm (1 3/8") bits.

Fig. A.21 – Pull-out curves of bolts installed in drill holes of different diameters. From Ames (A.12)



b) Tests in holes drilled with 31.75 mm (1 1/4") bits.

Fig. A.21 – (cont'd)



c) Tests in holes drilled with 30.9 mm (1 7/32") bits

Fig. A.21 – (cont'd)

Test	Diameter of the hole (in)	Installed load torque (lb–ft)	Initial tension (tonnes)	Load exceeding the yield strength (tonnes)	Anchor slip (in/1,000 lb)
1	1.450	85	1	T.I. (2.7)	0.307
2	1.435	100	1	T.I. (1.4)	0.450
3	1.410	165	1.9	T.I. (3.5)	0.154
4	1.275	130	3.3	9	0.026
5	1.275	85	1.4	9	0.034
6	1.280	315	3.2	9	0.021
7	1.225	100	1.7	9	0.067
8	1.268	175	3.6	T.I. (6.3)	0.045
9	1.250	280	4.5	9	0.026

T.I.: Test interrupted before exceeding the limit of elasticity (load reached, tonnes)

c) Stelco Tests (A.13)

This company has carried out numerous bolt pull-out field tests for the last few years. The tests have been conducted mainly in Ontario mines, as the result of the introduction of the above-mentioned regulation and to verify the performance of different expansion shells marketed by the company. Numerous tests were performed using a multiple-use anchor that can be used for holes with diameters of 1 1/4" to 1 3/8". This product has been recently introduced to solve the problem of drill holes that exceed the low drilling tolerances allowed for the other shells. The table in Figure A.22 summarizes the results of these tests, all of which were carried out in the same mine. The means and standard deviations for anchor slip obtained for each diameter are summarized below.

Drill hole diameter (in)	1 15/64	1 1/4	1 3/8
Anchor slip (in/1,000 lb)	0.024	0.036	0.045
Standard deviation	0.014	0.0005	0.021

The following conclusions may be derived concerning the multi-purpose anchor tested:

- All the tests were carried out up to a load close to or exceeding the yield strength of the bolt barrel (8 tonnes), including bolts producing the highest anchor slip values (0.05"/1,000 lb or more).

- In general, this shell does not seem to comply with the allowable slip limit of 0.028"/1,000 lb. However, we should point out that its performance in the 1 1/4" holes was much more consistent (low standard deviation) than in the 1 3/8" and 1 15/64" holes. The high standard deviation for the shells installed in the latter holes suggests that the shell cannot be inserted in holes where the diameter is beyond this limit of 1 15/64" without risking a wide range of performances.

Diameter of the drill hole (in) (as measured)	Installed load torque (lb–ft)	Initial tension (tonnes)	Load exceeding the yield strength (tonnes)	Anchor slip (in/1,000 lb)
1 15/64	175	2	T.I. (7.0)	0.010
1 15/64	175	3.5	T.I. (7.0)	0.028
1 15/64	175	2.8	T.I. (7.0)	0.042
1 15/64	190	2	T.I. (7.0)	0.018
1 1/4	225	4.3	8.5	0.037
1 1/4	240	4.2	8.5	0.036
1 1/4	225	3.8	8.5	0.037
1 1/4	200	3.8	8.5	0.036
1 1/4	225	6.2	8.5	0.036
1 3/8	-	1.0	T.I. (7.0)	0.010
1 3/8	-	1.3	T.I. (7.0)	0.021
1 3/8	-	3.5	T.I. (7.0)	0.068
1 3/8	225	4.2	8.5	0.032
1 3/8	250	4.7	8.5	0.045
1 3/8	195	3.3	8.5	0.053
1 3/8	220	3.8	8.5	0.083
1 3/8	180	4.3	T.I. (8.0)	0.078
1 3/8	140	6.0	8.0	0.061
1 3/8	150	4.5	8.0	0.045
1 3/8	185	5.5	8.0	0.016
1 3/8	195	6.0	8.0	0.046
1 3/8	250	5.0	8.0	0.022
1 3/8	160	2.5	8.5	0.035
1 3/8	185	4.2	8.5	0.042
1 3/8	180	4.5	T.I. (7.5)	0.057
1 3/8	160	5.5	8.5	0.045

T.I.: Test interrupted before exceeding the limit of elasticity (load reached, tonnes)

Fig. A.22 – Results of pull-out tests on bolts equipped with a multi-purpose shells for holes with diameters between 31.75 mm (1 1/4") and 34.9 mm (1 3/8"). From Lachapelle (A.13)

A.12 – COMPARISON OF COMBINATION AND MECHANICALLY ANCHORED BOLTS

The tests discussed were carried out by the Mines Safety and Health Administration (MSHA), to certify combination bolts for use as reinforcement in underground mines (A.15, 59).

Two types of tests were carried out: conventional pull–out tests on combination bolts with various grouting lengths and on mechanically anchored bolts, and tests to verify the loss of tension in combination bolts after 24 hours.

a) Results of the Pull–out Tests

The high anchoring capacity of bolts grouted over a short length has been verified by numerous laboratory and field tests. Figure A.23 shows average load curves obtained with 19 mm (3/4") bolts grouted with resin over a distance of 0.3 m (12") and 0.45 m (18") in Indiana limestone blocks (with a relatively low compressive strength of 50 MPa), as well as the average load curve for 15.9 mm (5/8") mechanically anchored bolts. In each case, the load was applied within five minutes after installation. Considering that the elongation of the bolt barrel was obtained from graphic data, the combination bolts were much stiffer than the mechanically anchored bolts.

The real advantage of the greater anchoring capacity of the combination bolts should in fact be realized in very soft rocks. Figure A.24 shows the results of tests carried out in a very soft clay schist where mechanically anchored bolts systematically slip with a pull–out force of about 26.6 kN (6,000 lb); the combination 15.9 mm (5/8") bolts could be loaded to the level of the yield strength as soon as five minutes after installation.

b) Results of Tests to Verify Loss of Tension After 24 Hours

These tests were carried out in an underground mine with bolts grouted with resin over a distance of 0.3 m (12") and tightened within 60 seconds after the resin was mixed.

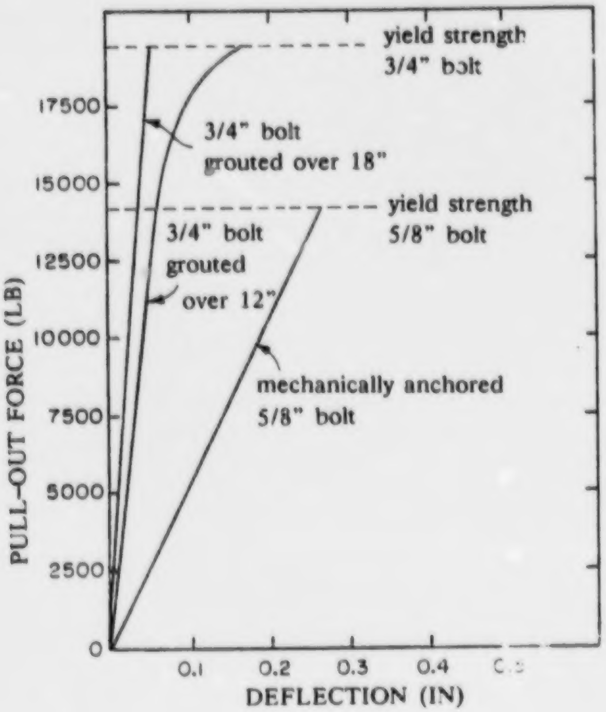


Figure A.23 – Pull-out curves of combination and mechanically anchored bolts in Indiana limestone. From Karabin *et al.* (A.15)

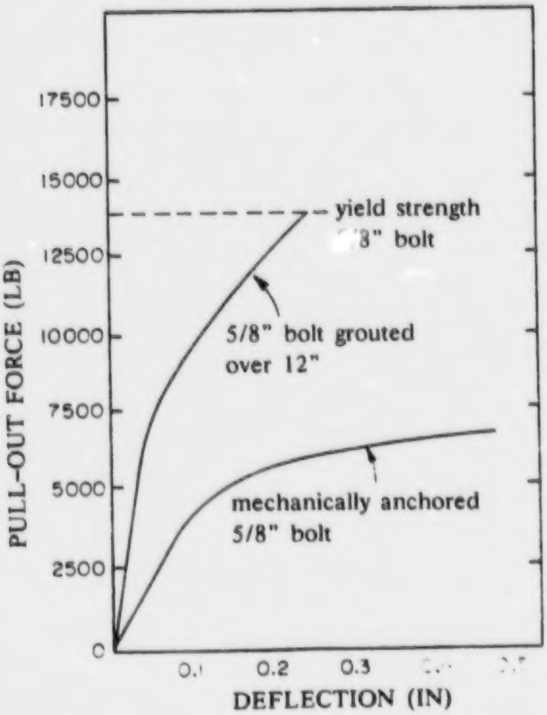


Figure A.24 – Average pull-out curves of combination and mechanically anchored bolts in a very soft clay schist. From Karabin *et al.* (A.15)

The results are summarized in the table below.

Bolt	Tension (lb)		Loss (%)
	At installation	After 24 h	
1	7,800	6,600	−15
2	6,600	4,100	−38
3	7,400	8,200	+11
4	7,000	6,200	−11
5	6,200	4,900	−21
6	6,200	5,900	−5
7	5,700	4,900	−14
8	7,000	5,300	−24
9	7,800	5,300	−32
10	6,600	5,500	−17
11	9,000	9,000	0
12	8,600	8,200	−5
13	6,600	6,200	−6
14	5,700	5,700	0
15	6,200	6,200	0
16	6,200	4,100	−34

The average loss observed was 13%. Although it is a significant loss, it is nevertheless about the same as the average loss observed for mechanically anchored bolts (see Appendix 6).

A.13 - DEMONSTRATION OF THE HOLDING POWER MECHANISM IN BOLTS GROUTED OVER THEIR ENTIRE LENGTH

This test was carried out by Serbousek *et al.* (A.16) on a bolt equipped with strain gauges (Figure 34, Section 3.2.7.1) and grouted with gypsum in a drill hole with a diameter of 25.4 mm (1"). The bolt was 1.2 m (48") long and had a diameter of 19 mm (3/4").

The pull-out load was applied to the head of the bolt using a jack-screw pulling device. The gauges were placed at intervals of 15 cm (6") and were read after each loading increment of about 8.4 kN (1,900 lb) to a maximum of about 56.9 kN (12,800 lb).

The bolt tensions measured in the location of each gauge and after each loading increment are shown in Figure A.25. This figure shows that the bolt tension dropped to zero at a distance of 0.57 m (22.5") from the head of the bolt, regardless of the load increment reached.

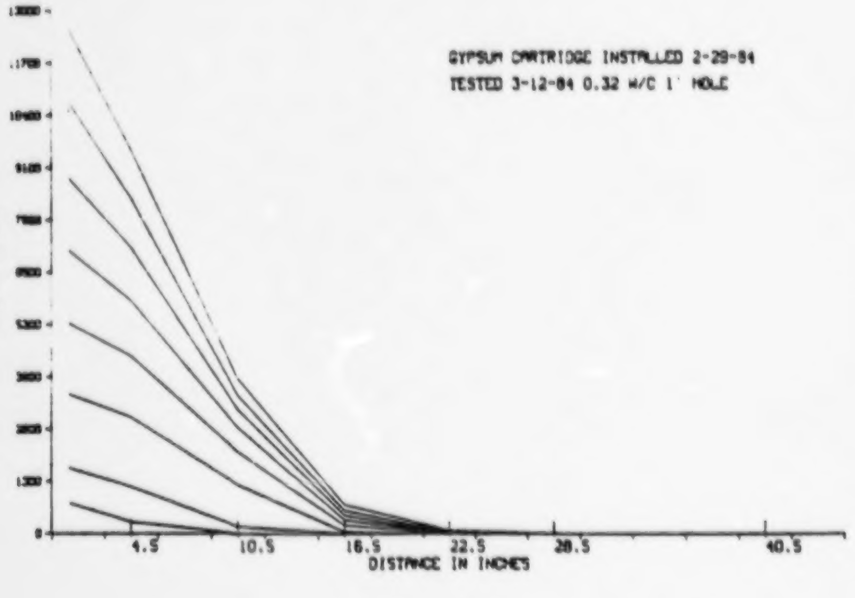


Fig.A.25 - Tensions measured in a bolt equipped with strain gauges and grouted with a gypsum cartridge. From Serbousek *et al.* (A.16)

However, the most important conclusion regarding the mechanism of the mobilization of holding power can be reached by observing that the slope of the curve varies depending upon the loading increment. If the holding power mechanism were solely the result of chemical adhesion (Section 3.2.4), the curves would be parallel, and the distance where tension drops to zero would increase with the force applied. As we can see, this phenomenon does actually take place at the 8.8 kN (2,000 lb) increment. Nevertheless, the main mechanism is the interlocking of irregularities in the bar–grout and grout–rock interfaces, which is most evident at the highest loading increments. In support of this mechanism, Serbousek *et al.* pointed out that when they removed the enclosing block, there was no evidence of cohesion or adhesion in the interfaces.

These results confirm the favourable effect on holding power that is obtained when the holes are drilled by percussion so that the walls of the hole are rough.

A.14 – RELATIONSHIP BETWEEN THE DIAMETER OF THE DRILL HOLE AND THE PULL-OUT FORCE OF RESIN-GROUTED BOLTS

These tests were carried out in 1976 by the Mining Enforcement and Safety Administration (MESA), U.S. Dept. of the Interior (A.14) on a large number of ribbed bars with a diameter of 19 mm (3/4"), grouted with resin over a distance of 0.3 m (12"), and installed in drill holes with the following diameters: 25.4 mm (1"), 28.6 mm (1 1/8"), 31.75 mm (1 1/4"), and 34.9 mm (1 3/8").

The average loading curves obtained are shown in Figure A.26. The following performances were observed, bearing in mind that the load at rupture of the bars tested was about 106.6 kN (24,000 lb).

Diameter of the drill hole (in)	Difference with the diameter of the bar (in)	Pull-out force (lb)	Loss in relation to the load at rupture of the bar (24,000 lb) (%)	Stiffness (slope of the loading curve) (lb/in)	Comments
1 3/8	5/8	9,500	60	20,000	Shearing of the resin
1 1/4	1/2	22,500	10	45,200	Shearing at the rock-resin interface
1 1/8	3/8	24,000	0	50,500	Rupture of the bolt barrel
1	1/4	24,000	0	400,000	Rupture of the bolt barrel

Bolts grouted in 1" and 1 1/8" holes produce acceptable performances. Nevertheless, the 25.4 mm (1") hole showed stiffness values that were nine times higher than those of 28.6 mm (1 1/8"), which represents an advantage in most situations, except when there is a significant

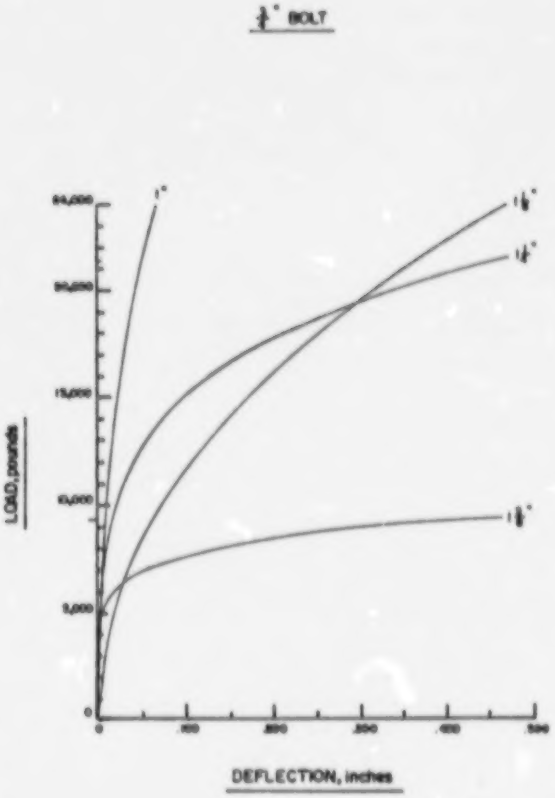


Figure A.26 - Pull-out curves for 19 mm(3/4") diameter bolts grouted with resin in drill holes of various diameters. From Karabin *et al.* (A.14)

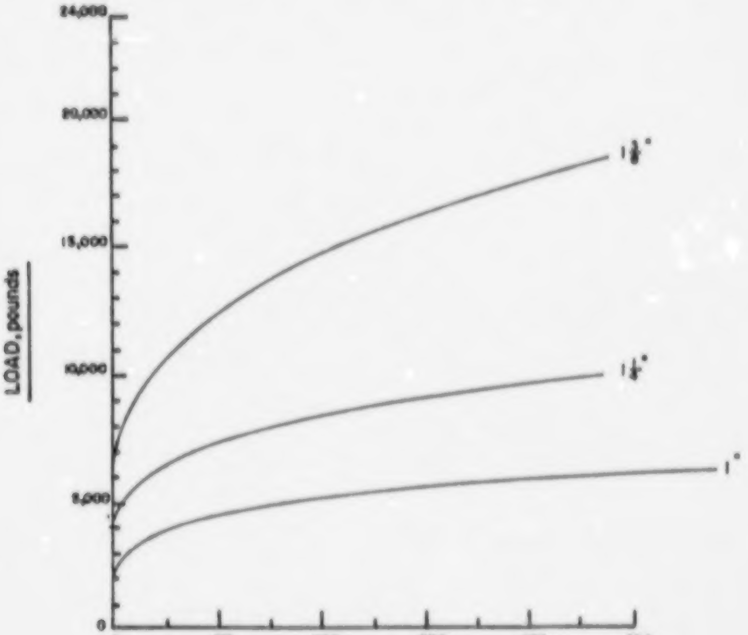


Figure A.27 - Pull-out curves of resin-grouted bolts in a low strength rock-like material. From Karabin *et al.* (A.14)

amount of ground convergence. The main conclusion that can be derived from this test is that the bolts must be grouted in drill holes with diameters that exceed by 6.4 mm (1/4") the diameter of the bolt itself. A diameter difference smaller than this cannot be recommended because of the inevitable irregularities that result when drilling the holes. Also, the tests showed that the outer shell of the resin cartridges was always torn when this smaller diameter difference was maintained, while a larger diameter difference could counteract this tearing.

Bolts grouted in 31.75 mm (1 1/4") holes already showed a significant loss of strength. Both the holding power and stiffness values of those grouted in 34.9 mm (1 3/8") holes were too low.

Acknowledging that the use of 19 mm (3/4") bolts in 25.4 mm (1") holes represents a significant saving, MESA suggests also that for excessively soft rocks the bolts used should have a larger diameter and should always be installed in drill holes with diameters that are more than 6.4 mm (1/4") larger. Increasing the diameter for excessively soft rocks makes it possible to have a larger contact surface between the resin and the wall of the drill hole, and thus to increase the pull-out force. The pull-out forces shown in Figure A.27 were obtained with 0.3 m (12") bolts with diameters of 19 mm (3/4"), 25.4 mm (1"), and 1 1/8", grouted in holes measuring 25.4 mm (1"), 31.75 mm (1 1/4"), and 34.9 mm (1 3/8") respectively, in a sand and mortar material with a compressive strength of 700 psi (4.8 MPa). The observed increase in holding power was considerable, from 26.6 to 80.0 kN (6,000 to 18,000 lb), while stiffness increased from 1,355 to 4,065 N-m (12,000 to 36,000 lb/in). The rupture load of the bars could not be reached during these tests, mainly because the grouting length was so short and the material had such little strength.

A.15 – EFFECT OF MINERAL AND CHEMICAL ADDITIVES ON THE HOLDING POWER OF CEMENT–GROUTED BOLTS

This study was carried out by Ballivy *et al.*(A.17) using Dywidag bars with a diameter of 34.9 mm (1 3/8") (yield strength of 850 MPa, load at rupture of 1,000 kN, grouted in holes with a diameter of 76.2 mm (3") (63.5 mm (2 1/2")) mm for resin–grouted holes) and drilled into a volcano–detrital rock mass with a very high compressive strength (212 MPa).

The grouting products used were an epoxy resin or conventional cement cartridges to which the following products were added or used to substitute for part of the cement: expansion agent (aluminum powder), superplastifier, silica ashes, and Ottawa sand (ASTM C–109). In all cases, the cement used was type 10 Portland. The exact composition of these products is shown in the table in Figure A.28.

Grout C_1 was a conventional mixture with a water/cement ratio of 0.4. Grout C_2 was an expandable mixture made like C_1, but to which aluminum powder was added in a proportion of 0.05 g per 1,000 g of cement. In grout C_3, 10% of the weight of the cement used for C_2 was replaced by the same amount of silica ashes. Grout C_4 was made by adding to grout C_3 a weight of sand equal to the total weight of the cement and silica ash, to obtain a grout that would show less shrinkage.

The viscosity of the grouting products containing silica ash (C_3 and C_4) was adjusted to the same value as that of products C_1 and C_2 by adding a naphthalene–based superplastifier in a proportion of 2.9 g per 1,000 g of cement.

The pull–out tests were carried out on bars grouted over various distances from 250 to 700 mm. As a general rule, rupture was located at the bar–grout interface. Nevertheless, the average shear stresses at rupture shown in the table in Figure A.29 were changed to average stresses acting on the grout–rock interface. The 28 day compressive strength of the various grouting mixtures also appear in the table.

	C_1	C_2	C_3	C_4
Water (W)	1	1	1	1
Portland cement type 10 (C)	2.5	2.5	2.25	2.25
Silica ash (Si)	–	–	0.25	0.25
W/(C + Si)	–	–	0.4	0.4
Ottawa sand	–	–	–	2.5
Aluminum powder	–	1.25×10^{-4}	1.25×10^{-4}	1.25×10^{-4}
Superplastifier	–	–	0.65×10^{-2}	0.65×10^{-2}

Note: The proportion of each component was calculated as a function of the weight of the water.

Fig. A.28 – Composition of cement–based grouting mixtures. From Ballivy *et al.* (A.17)

Grouting mixture	Compressive strength (MPa)	Number of tests (MPa)	Shearing stress at rupture*	Number of tests
R	110 ± 5.1	6	7.3 ± 0.41	9
C_1	64.3 ± 2.5	6	5.8 ± 0.08	2
C_2	49.5 ± 3.2	10	6.3 ± 0.1	5
C_3	59.6 ± 1.7	6	7.2 ± 0.15	2
C_4	66.3 ± 2.7	6	8.7 ± 0.18	6

R: epoxy resin (type 1530), Fastloc–T, made by Dupont Canada

* mean stress at the grout–rock interface

Fig. A.29 – Pull–out strength of Dywidag bars as a function of the grouting product used. From Ballivy *et al.* (A.17)

The following observations could be made from the table:

- The holding power of the epoxy resin (7.3 MPa) is lower than that of cement-based products, except in the case of grout C_1 (5.8 MPa). However, the compressive strength and other mechanical characteristics of this product are clearly superior. This can be attributed to inevitable grouting imperfections (poor contact between surfaces, empty spaces, pieces of cartridge plastic).

- Grout C_2 showed an improvement in pull-out resistance or shear stress of 0.5 MPa, even though its compressive strength is lower than that of the reference grout (C_1). This reduction is attributable to voids created by the hydrogen released because of the chemical reaction between the aluminum powder and the chalk, which separates as the result of the hydration of the cement. In this case, as for grouts C_3 and C_4, the improvement in strength can be explained by the use of the aluminum powder, which leads to the confinement of the grout as the result of expansion caused by the release of hydrogen. The cracks caused by shrinkage of the grout are smaller, and the contact between the grout, the surface of the bar, and the walls of the drill hole is also improved.

- The silica ashes contained in grout C_3 caused an improvement in pull-out resistance by 1.4 MPa as the result of an increase in the compressive strength of the grout, and a decrease in its porosity.

- The sand contained in grout C_4 produced a very significant strength improvement of 2.9 MPa. The incompressibility of the sand acts at two levels: it reduces the shrinkage of the grout, which leads to an improvement in adhesion; and it produces a volume expansion at rupture, which leads to an increase in friction strength.

- The results of these tests show that the compressive strength of the grouting mixture is not the only parameter that should be taken into consideration in order to obtain better pull-out resistance. Factors such as confinement and low shrinkage also play important roles.

A.16 - RESULTS OF PULL-OUT TESTS ON SWELLEX BOLTS INSTALLED IN SOFT ROCK

The results of the tests reported below were obtained by the Atlas Copco company (A.18) in various mines or civil engineering installations. Only those tests performed in soft or very soft rocks, almost comparable to soils, are reported.

The results are shown in Figure A.30. In all cases, except where indicated otherwise, the bolts slipped and it was impossible to attain the nominal load at rupture of the bolt (100 kN). Very often, this problem occured because the bolts tested were too short.

Type of rock	Diameter of the drill hole (mm)	Swelling pressure (MPa)	Length of the bolt (m)	Slip load (kN)	Unit slip load (kN/m)
Serpentinite	32	30	1.53	71	46
Sericite schist. chlorite	32	30	1.36	97	72
Peridotite	32	30	1.48	97	66
Soft rock 6.9 MPa (1,000 psi)*	35	25	0.76	22	29
Soft rock 27.6 MPa (4,000 psi)	38	28	0.91	132	145
Soft rock 4.1 to 6.9 MPa (600 to 1,000 psi)	32	22	0.91	43	47
Soft rock 34.5 MPa (5,000 psi)	41	29	1.06	41	39
Sericite schist. chlorite	38	30	0.45	75	166
Soft tertiary sandstone 40 to 80 MPA (5,800-11,600 psi)	35-38	30	1.3	115 (rupture)	88
Stiff clay	37	30	0.36	44 to 102 (at installation)	16 to 37
				57 to 108 (after 5 days)	20 to 39
				71 to 93 (after 6 weeks)	25 to 33

* compressive strength of the rock

Fig. A.30 - Results of pull-out tests on Swellex bolts used in soft rock (A.18)

A.17 - COMPARISON OF RESIN-GROUTED AND SPLIT SET BOLTS INSTALLED IN THE ROOF OF GALLERIES IN BEDDED FORMATIONS

These tests were carried out on behalf of the U.S. Bureau of Mines (A.19) in the underground mine of the White Pine Copper Company in Michigan. The purpose of the tests was to compare the performances of resin-grouted and Split Set bolts in an environment similar to that encountered in coal mines, that is, bedded formations with alternating siltstones and clay schists above the mineralized copper formation. Moreover, it is generally accepted that grounds at the mine are affected by high horizontal stresses, about 55 MPa (8,000 psi) at the 375 m deep test site, which contribute to the loosening of layers in the roof of the galleries. The mine is exploited by the room-and-pillar method.

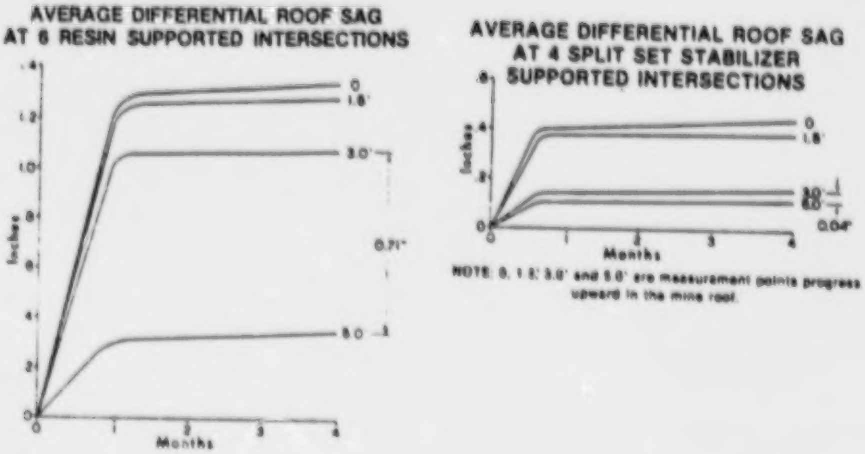
A total number of 625 SS-39 Split Set bolts and an equivalent number of resin-grouted 19 mm (3/4") bolts were installed in holes with a diameter of 25.4 mm (1"). The bolts were 1.2 m (4 ft) long and were installed in accordance with the normal pattern used in the mine, 1.2 x 1.2 m (4 ft x 4 ft).

Pull-out tests were carried out on some of the bolts. The results obtained showed that the stiffness of the resin-grouted bolts was much higher than that of the Split Set bolts. The resin-grouted bolts produced a holding power of about 350 kN/m (2,000 lb per in.) and the split set bolts of 22 to 52 kN/m (125 to 300 lb per in.).

Nevertheless, for these tests, it did not seem desireable to have a very stiff support. In fact, several gallery intersections, as well as various points located 0.45 m (1.5 ft), 0.9 m (3 ft), and 1.5 m (5 ft) in the rock mass, were equipped with devices to measure the amount of sagging in the roof of the gallery.

The results of these measures are shown in Figure A.31.

The total amount of sag in the roof (depth 0 m) was 35.5 mm (1.4") with the resin-grouted bolts and did not exceed 10.2 mm (0.4") with the Split Set bolts. An explanation of this phenomenon can be derived from the same measurements.



AVERAGE DIFFERENTIAL ROOF SAG
AT 6 RESIN SUPPORTED INTERSECTIONS

AVERAGE DIFFERENTIAL ROOF SAG
AT 4 SPLIT SET STABILIZER
SUPPORTED INTERSECTIONS

NOTE: 0, 1.5', 3.0' and 5.0' are measurement points progress upward in the mine roof.

Fig. A.31 - Roof sag in galleries supported by resin-grouted and Split Set bolts (A.15, 58)

In fact, we can see that between depths 0 m and 0.9 m (0 ft and 3 ft), the sag was similar in both cases, about 7.6 mm (0.3"). The difference is the result of the sag caused by the separation of layers between the 0.9 m and 1.5 m (3 ft and 5 ft) depths, which was 18 mm (0.71") for sections supported with resin-grouted bolts, and only 1 mm (0.04") for those supported with Split Set bolts. Moreover, a study of the stratigraphy of the roof in the test location showed that the first 1.2 m (4 ft) of the roof (the thickness equivalent to the length of the bolts) contained thinly bedded formations, and that these formations were in contact with a more massive 0.45 m (18")bed immediately above.

Thus, the increased stiffness of resin-grouted bolts would have contributed to the creation of a rupture line precisely at the boundary of the bolts; the Split Set bolts, which are not as stiff, would have contributed to the gradual development of a state of equilibrium in the ground and prevented the formation of this rupture zone. Moreover, this effect is similar to the formation of a natural rock vault that was discussed in Section 2.3.1.

Nevertheless, it was also acknowledged that the perimeter blasting techniques used exclusively in the test areas supported with Split Set bolts also played a role in the smaller amount of sagging observed in these areas.

A.18 – AVERAGE DENSITIES OF BOLTING OBSERVED IN 13 UNDERGROUND MINES IN QUEBEC

In 1984, data was collected by the author in 13 underground mines concerning the number of bolts consumed in the galleries and the amount of annual development, as well as information on the depth and width of the galleries excavated (16).

This information was used to calculate the average density of rock bolting used, that is, the number of bolts used per square metre of gallery roof, and gallery walls if these were also bolted. This value was obtained by dividing the number of bolts used during the year by the number of gallery metres advanced, and then by the perimeter length of the gallery where the bolts were installed (roof only or roof and walls, depending on the case).

Example: 15,000 bolts per year for 200 metres of 3.5 m x 5 m gallery, where both the roof and walls are bolted.

$$\text{Density of bolting: } \frac{15{,}000}{2{,}000\ (3.5 + 5 + 3.5)} = 0.625 \text{ bolts/m}^2 \text{ of roof and wall}$$

Note: To progress from the number of bolts per square metre to the installation pattern of the bolts, we must take the inverse of the square root of the first number. In the previous example, this operation produces:

$$\frac{1}{\sqrt{0.625}} = 1.26$$

Thus, the installation pattern is a grid measuring 1.26 m x 1.26 m.

The results of the survey are shown in Figure A.32. The figures for the annual consumption of bolts shown on the table were obtained from various sources, such as the mine storage inventory, and the productivity bonus records. In both cases, the figures obtained could have been slightly overestimated. Ten percent was often mentioned during our visits.

Mine	Main exploitation depth (m)	Most Common gallery size (height x width)	Annual consumption of bolts in the galleries (excluding stopes)	Annual gallery advance (m)	Number of bolts per square metre of roof (and of walls, if applicable) (m)	Equivalent installation grid for roof bolts (and walls, if applicable)
1	330	3m x 3 m	1.800 (7 months)	1.693 (7 months)	0.36 (roof)	1.66m x 1.66 m
2	135	2.4m x 2.4 m	720	2.500	0.12 (roof)	2.88m x 2.88 m
3	175	2.7m x 2.4 m	1.500 (8 months)	3.600 (8 months)	0.17 (roof)	2.42m x 2.42 m
4	70	2.8m x 4 m	-	-	0.55 (roof)	1.35m x 1.35 m
5	80	3.6m x 4.5 m	4.800	1.098	0.98 (roof)	1.01m x 1.01 m
6	600	2.4m x 2.4 m	25.100	8.738	1.10 (roof)	0.95m x 0.95 m
7	300	3.5m x 4.9 m	22.000	4.390	1.02 (roof)	0.99m x 0.99 m
8	330	3.5m x 5 m	18.400	1.465	1.48 (roof and 2 rows in the walls)	0.82m x 0.82 m
9 (waste)	300	3.5m x 3.5 m	33.500	3.974	0.08 (roof and walls)	1.12m x 1.12 m
9 (ore)	300	3.5m x 4 m	21.950	1.772	1.13(roof and walls)	0.94m x 0.94 m
10	350	3m x 4 m	1.750 (1 months)	385	1.14 (roof)	0.94m x 0.94 m
11	850	3.3m x 4 m	6.350 (8 months)	1.400	0.75 (roof and 1 row in the walls)	1.15m x 1.15 m
12	750	3m x 5 m	2.5500 (1 months)	2.912	1.62 (roof)	0.78m x 0.78 m
13	290	3m x 3.6 m	6.240 (5 months)	368	1.76 (roof and walls)	0.75m x 0.75 m

Fig. A.32 – Results of a survey on the number of bolts used in Quebec mines in 1984 (16)

The following observations can be made from the table:

- Mines 1 to 4 use a low density of bolts per square metre, in the order of 0.12 to 0.17 for mines 2 and 3 (equivalent installation pattern of 2.88 m x 2.88 m to 2.42 m x 2.42 m), and 0.36 to 0.55 for mines 1 and 4 (average installation pattern of 1.66 m x 1.66 m to 1.35 m x 1.35 m). Of the 13 mines, these are the only four where the amount of gallery support is decided in accordance with need (no set bolt installation pattern); moreover, they are the smallest mines visited, with a daily output of less than 600 tonnes. Mines 2 and 3 had the smallest gallery dimensions (width, 2.4 m) of all the mines visited, and both operate at shallow depths (less than 175 m). The density values of 0.2 and 0.17 were quite close, which shows that mines operating under conditions that could be described as "**light**" (depth of 30 m to 175 m, gallery width of about 2.7m) tend to have a bolting density in accordance with a density of bolting of 0.2 bolts/m² (installation grid of 2.25 m x 2.25 m).

- The density values of 0.36 and 0.55 in mines 1 and 4 are between the previous values and those of mines 5 to 13, which are discussed below. The bolting density in mine 1 was certainly lower than bolting densities of all the other mines also at depths of about 300 m. This can probably be explained by the 3 m width of the galleries, relatively narrow in comparison with galleries in mines 5 to 13. However, it would be preferable to increase the density of bolting to 0.5 in this mine, to bring it closer to a midpoint between the trends in mines 2 and 3, and mines 5 to 13. For mine 4, the density of 0.55 can be explained by the shallow depth (about 70 m), which compensates for the width of the galleries (4 m).

  The experience with these two mines (1 and 4) shows that mines operating under conditions similar to theirs (conditions that could be described as "**intermediate**": depth of 175 to 500 m, and gallery width of about 2.7 m) use a bolting density of 0.6 bolts/m² (installation grid of 1.3 m x 1.3 m).

- Mines 5, 6, 7, 9, and 10 had very similar bolting densities per square metre: between 0.98 and 1.14, which correspond to installation grids of 1.01 m x 1.01 m to 0.94 m x 0.94 m. These five mines use a density of bolting of about 1.0 and have two elements in common: their depth, which is close to 300 m, and the size of the galleries, which measure between 4 and 5 m in width. This observation shows that, of the 13 mines visited, a large number (five) have developed the same bolting density and it indicates that mines operating under similar conditions (those that could be described as "**normal**": depth of 175 to 500 m, gallery width of 4 to 5 m) use a density of bolting of 1.0 bolt/m² (installation grid of 1 m x 1 m).

- The following points regarding the five mines could be emphasized. Mine 5, which has a density of 0.98, operates under conditions that were previously described as intermediate because it is not as deep, about of 80 m. This density, which is higher than what could be considered normal in other mines operating under intermediate conditions, occurs because the two levels exploited are very close to the surface, at depths of 20 m and 40 m. Mine 6 has a density of 1.10 even though the galleries are small (2.4 m x 2.4 m). On the other hand, it reaches 600 m in depth, which gives another example of what could be considered "normal" support conditions (depth of 500 to 750 m, gallery width of 2.7 m), for which we observed a density of 1.0. Mine 9, where the ground conditions are very difficult, uses a density of 1.13 for sub-levels in the orebody with a width of 4 m and a density of 0.80 for haulage ways 3.5 m wide in the hanging-wall. This narrower gallery width helps explain the lower density of 0.80. All other observations in this mine, which operates under "normal" conditions (depth of 300 m and galleries 4 m wide), suggest the density of bolting observed in Quebec mines is mainly influenced by two parameters, depth and gallery width, rather than by the general (and not the local) ground conditions in the mine.

- Mine 11 uses a bolting density of 0.75, which seems low for its depth (850 m) and the width of its galleries. Its true density is probably higher, about 1.0, because several galleries were narrower than the 4 m gallery that was used for the calculation.

- Mines 8, 12, and 13 have high bolting densities, ranging from 1.48 to 1.76 bolts/m² (average installation pattern of 0.82 m x 0.82 m to 0.75 m x 0.75 m). In mine 8, the high density can be explained by the relatively short length of the bolts in relation to the width of the galleries.

- Mine 13 uses a high bolting density (1.76), even though it operates under "normal" conditions. However, as in the case of mine 9, bolting in mine 13 is used to control convergence of the walls. Mine 9 controls this problem successfully with a density of

1.13, but mine 13 adds a final shotcrete coating for which the resistance to wall convergence is limited.

- Mine 12 is the only one of the mines visited to open galleries under conditions that could be described as "reinforced": depth of 750 m and galleries 5 m wide. The density of bolting in this mine reaches 1.62, which could indicate that mines operating under such **"reinforced"** conditions (depth of 500 to 750 m, galleries 4 to 5 m wide) use a density of bolting in the order of 1.5 bolts per square metre (installation grid of 0.8 m x 0.8 m).

A.19 – CALIBRATION OF THE SCHMIDT HAMMER ON IGNEOUS ROCKS OF THE CANADIAN SHIELD*

a) Previous Studies Carried Out on the Schmidt Hammer

In 1948, the Swiss engineer Ernest Schmidt developed a sclerometer that made it possible to establish a relationship between the uniaxial compressive strength of concrete and a rebound index that could be read on the device.

Subsequently, many authors have suggested the use of the Schmidt hammer to evaluate the uniaxial compressive strength of intact rock (σ_c) for engineering purposes. An excellent summary of these studies has been carried out by Haramy *et al.* (A.20). This summary discusses the main formulas that can be used to evaluate compressive strength.

A review of studies carried out by the different authors clearly shows that the value of (σ_c) obtained with these formulas is impaired by a relatively high scatter. This scatter results from the heterogeneous nature of the rock and also from the testing procedure, which must be followed very carefully. When testing rock core samples, it is particularly important to attach them to a metallic base to dampen the vibrations. In spite of this precaution, the results are frequently widely scattered; scatter is often worse when the tests are carried out on larger blocks of rock.

The purpose of the calibrations presented in this appendix is precisely to discourage the use of the Schmidt hammer to determine the compressive strength of rock core samples, since our own tests produced results that were too widely scattered to consider that the method is of any practical interest. On the other hand, the results that we obtained on large rock blocks, as well as those obtained in the walls of the galleries, were more significant. This will be discussed below.

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*The calibration work reported here was carried out by Charette (17) and the author.

For purposes of calibration, a single formula, that of Deere and Miller (A.21), was used, as follows:

$$\sigma_c = 6.9 \; x \; 10^{(0.16 + 0.0087 . R . \gamma)}$$

where R: rebound index measured with a type L hammer

γ: density of the rock (g/cm^3)

σ_c : in MPa

This formula is also recommended by the International Society of Rock Mechanics.

b) Test of the Schmidt Hammer on Rocks in the Canadian Shield

Two types of verifications were carried out.

The first consisted of testing the Schmidt hammer on seven blocks of rock weighing 75 kg or more, obtained from a copper and zinc mine in Northwestern Quebec.

Each block was subjected to the impact test in accordance with the following procedure: 20 consecutive impacts moving the hammer by 1 cm after each test, and then calculating the average rebound on the basis of the 10 highest values. Blocks were then drilled to obtain a certain number of core samples with a diameter of 50 mm. The density (γ) of the samples was determined before subjecting them to a conventional uniaxial compressive test in a loading frame.

The results of these tests are shown in Figure A.33, where the graph illustrates the relationship between (σ_c) and the product ($R \; x \; \gamma$). The equation of Deere and Miller is superimposed onto the graph using a continuous line.

In the figure, it would appear that the five blocks of rhyolitic and basaltic tuffs correspond closely to the trend proposed by the equation of Deere and Miller, even though the curve (indicated by a dotted line) passing through the points is offset by $R \; x \; \gamma = 7.5$. This shift could be explained by a scale effect that affects the result of tests carried out on large blocks, where the impact of the hammer is dampened in a volume of rock that is probably greater than that of rock core samples (like those tested by Deere and Miller), even if these samples are attached to a steel base. On the other hand, we believe that it is not likely that an additional scale effect would be added when the hammer is used on the wall of a gallery.

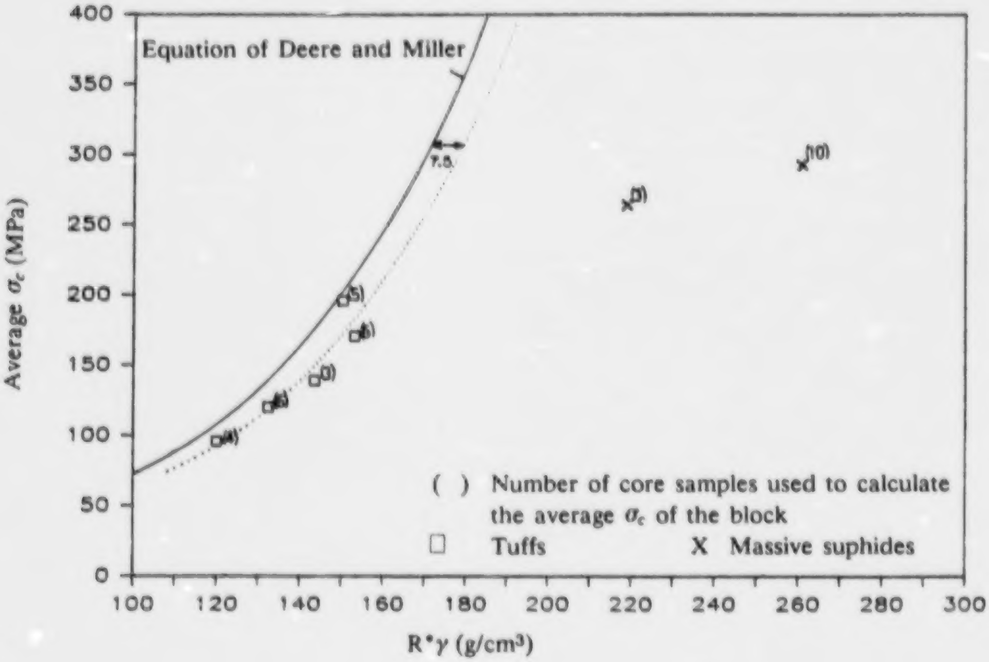

Fig. A.33 – Calibration of the Schmidt hammer on 75 kg rock blocks

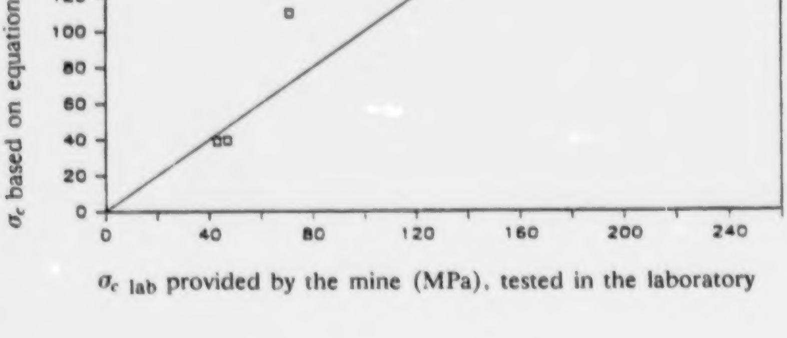

Fig. A.34 – Calibration of the Schmidt hammer on the wall of mine galleries

The two blocks of sulphide rock masses with a specific gravity of 4.0 produced points that fall outside the trend predicted by Deere and Miller. The situation is not surprising because, if we go back to the original studies carried out by these authors, we can see that the rocks tested to establish the equation always had a specific gravity of less than 3.2. Sulphide rock masses and other rocks with high specific gravities seem to represent a special case.

The second verification of the validity of the Schmidt hammer consisted of using it on the wall of the galleries visited during the measuring campaign carried out in 1985 in 10 Quebec mines. The rebound values obtained were used in the formula of Deere and Miller with $R_{modified} = R - 7.5$, and then compared with the value of the compressive strength of the rock provided by the mine ($\sigma_{c\ lab}$) and tested by conventional methods. The results of this operation are shown in Figure A.34, where it seems that the relationship between rebound and compressive strength is very close, taking into account that the values of compressive strength provided were not necessarily obtained from the same location as the rebound values in the mine.

c) Utilization Method of the Schmidt Hammer

We recommend using the Schmidt hammer to obtain the value of the compressive strength of the rock in a gallery. This value approximates the value that would be obtained if core samples were taken form the same location and used to carry out a conventional uniaxial compressive test or a point load test (Sections 4.2.1.1 and 4.2.1.2).

The procedure is as follows:

- Find the location to be tested; it must be free of any traces of dust.

- Apply the type L Schmidt hammer to the wall of the gallery, and carry out 20 impacts while moving the hammer at least 1 cm after each impact.

- Calculate the rebound index (R) by obtaining the average of the 10 highest readings.

- Correct the R index as a function of the inclination of the hammer, with the help of the table shown in Figure A.35. For example, when the hammer is held horizontally to hit the vertical wall of a gallery, and the value obtained is R = 50, the corrected value is R = 50 − 2.2 = 47.8.

- Calculate the uniaxial compressive strength (σ_c), using one of the following two methods:

  • If $\gamma \leq 3.2\ g/cm^3, \sigma_c = 6.9\ x\ 10^{(0.16 + 0.0087.(R.\gamma - 7.5))}$

  This is the formula of Deere and Miller modified to take into account the 7.5 shift observed on Figure A.33. The evaluation graph shown below can also be used.

  • If $\gamma > 3.2\ g/cm^3$

  Use the evaluation graph shown in Figure A.36. This graph was established based on the experimental points shown in Figure A.33. It takes into account the shift observed in the Deere and Miller equation.

Corrections for reducing measured Schmidt hammer rebound (r) when the hammer is not used vertically downwards

Rebound	Downwards		Upwards		Horizontal
r	$\alpha = -90°$	$\alpha = -45°$	$\alpha = +90°$	$\alpha = +45°$	$\alpha = 0°$
10	0	−0.8	—	—	−3.2
20	0	−0.9	−8.8	−6.9	−3.4
30	0	−0.8	−7.8	−6.2	−3.1
40	0	−0.7	−6.6	−5.3	−2.7
50	0	−0.6	−5.3	−4.3	−2.2
60	0	−0.4	−4.0	−3.3	−1.7

Fig. A.35 – Table of corrections for the R index as a function of the inclination of the Schmidt hammer (A.22)

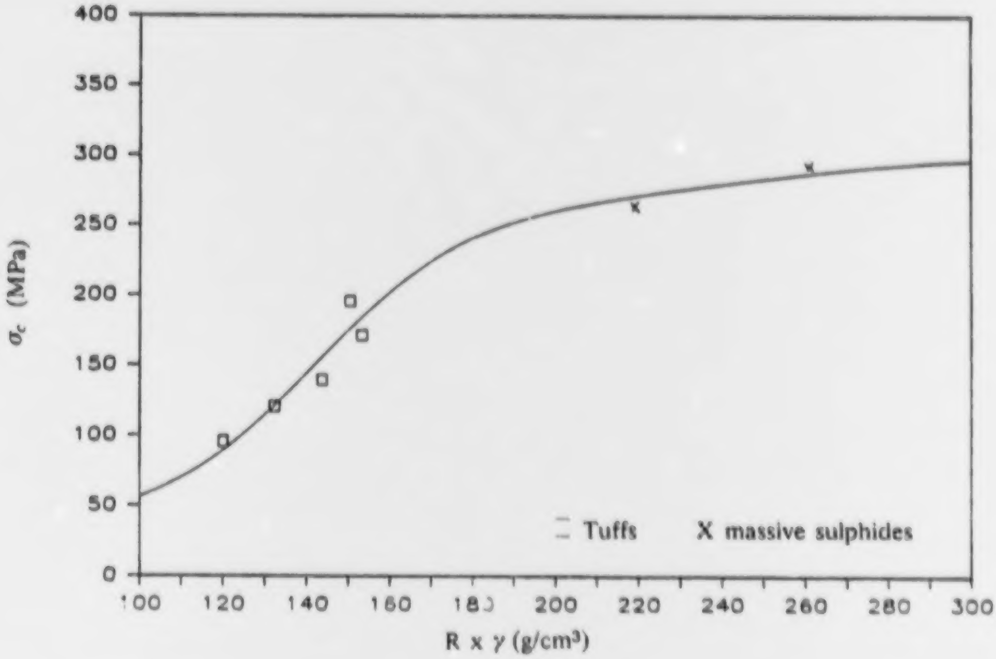

Fig. A.36 – Graph for the evaluation of uniaxial compressive strength for high density rocks with the Schmidt hammer

A.20 - VALIDATION OF A FORMULA TO ESTIMATE THE RQD OF IGNEOUS ROCKS IN THE CANADIAN* SHIELD

The formula used to estimate the RQD is as follows:

$$RQD = 100. (0.1\lambda + 1).e^{-0.1\lambda}$$

where λ: number of fractures per metre

This formula was proposed by Priest and Hudson (65). It derives from the calculation of the probability of finding spaces between fractures larger than 0.1 m (4"), when the spaces between fractures obey a negative exponential distribution law characterized by frequency λ. Consequently, to validate the formula used to estimate the RQD, it is necessary to confirm that the distribution of the spaces between fractures follows a law of the type $f(x) = \lambda e^{-\lambda x}$, whose curve is shown in Figure A.37. This verification is carried out in the next paragraphs.

a) Method Used to Verify the Negative Exponential Distribution

To carry out this verification, we surveyed spaces between fractures crossing thirty 15 m traverse lines during the measuring campaign carried out in 1985 in 10 Quebec mines. The surveys were conducted very accurately: the spaces were surveyed at the scale of almost 2.54 cm (1"). It is important to emphasize that this precision was necessary only for verifying the fit of a negative exponential law. In practice, when we want to use the relationship between the RQD and the number of fractures per metre λ, it is enough to count the number of joints crossed by the line and divide by its length to obtain the average spacing, and then to take the reverse to obtain the number of fractures per metre.

For each of the 30 traverses we calculated the average spacing ($\bar{x}$) of the joints, the standard deviation s, and the frequency λ (number of joints per decimetre). Because the number of observations was relatively low, between 12 and 151 observations per traverse, we considered that it would be appropriate to group the observations into 1 dm categories (0.1 m or 4") before processing the information.

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*The validation reported here was carried out by Charette (17) and the author.

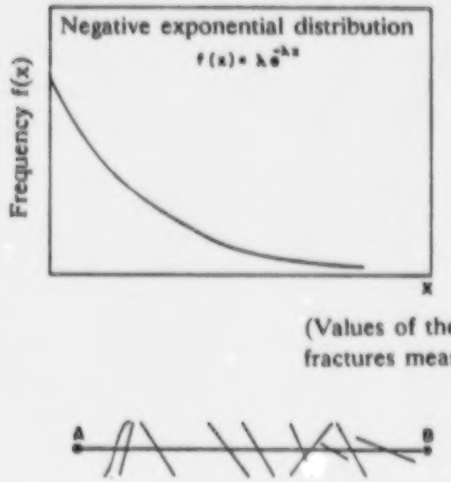

Negative exponential distribution
$f(x) = \lambda e^{-\lambda x}$
Frequency f(x)
x
(Values of the spacings between
fractures measured on traverse A–B)
A
B

Fig. A.37 – Theoretical shape of the distribution of spaces between fractures measured on a traverse

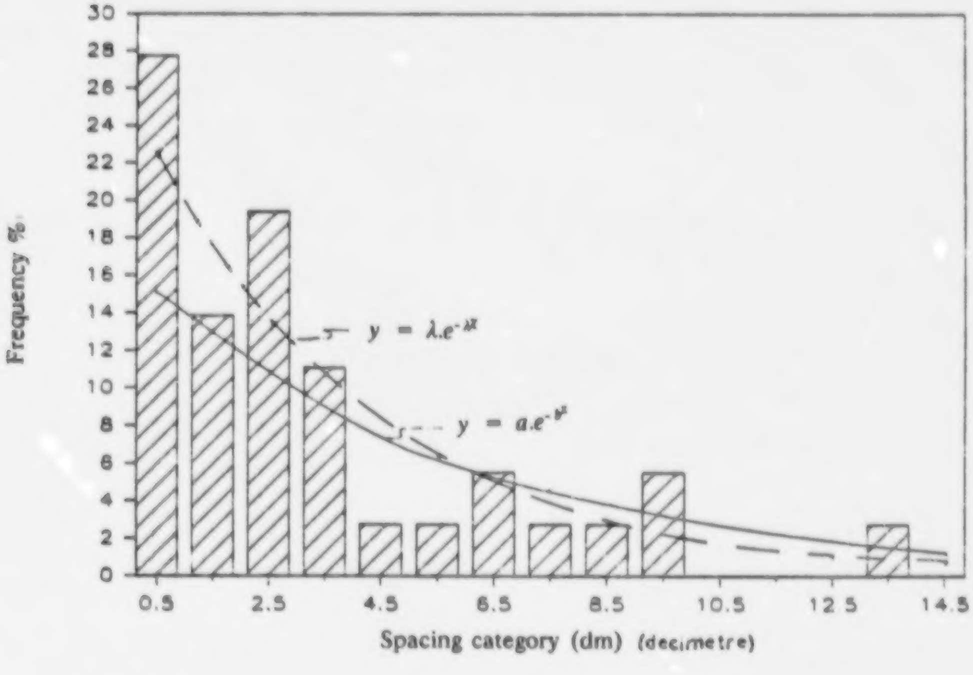


Fig. A.38 – Histogram of the spacing categories observed for traverse no. 3 (36 observations)

The theoretical distribution to be verified is continuous, while the category grouping described above produces a discrete distribution. Nevertheless, considering the midpoint value in each discrete category, we can also obtain a negative exponential curve, such as that shown in Figure A.38, for example.

Two separate operations were carried out on the frequencies of the 0.1 m spacing categories. The first was a fit using least squares of the midpoint of the categories to a formula of the type $f(x) = a.e^{-bx}$, where x represents the measured spacings. The second operation is a Chi-square test with μ degrees of freedom and a significance level of $1-\alpha$ of 95%; that is, (α) equals 0.05 for a formula of the type $f(x) = \lambda e^{-\lambda x}$, where x represents the measured spacings, and λ the average number of fractures per decimetre. In the latter case, we assumed that $\lambda = 1/\bar{x}$. On the other hand, the Chi-square test was carried out only for those traverses where the value of expression $\frac{b-a}{a}$ was less than 0.25. This acceptance criterion meant that the distribution observed was *a priori* sufficiently close to a negative exponential law where we would ideally have a = b.

The results of the fit by least squares are shown in the table in Figure A.39. The table shows the parameters already mentioned, as well as the coefficient of correlation r of the fit given for purposes of information, and the decision to accept or reject the traverse. At the same time, Figure A.40 shows the results of the Chi-square test, particularly the calculated values of Chi^2 and $\mathrm{Chi}^2_{0.05,\mu}$ critical for each of the traverses that were not rejected during the previous stage, as well as the final decision to accept or reject the traverse.

No. of the traverse	No. of observation	Average spacing $\bar{x}$ (dm)	Standard deviation s (dm)	Frequency λ (frac./dm)	"a"	"b"	"r"	(b−a)/b (%)	Acceptance
1.000	151.000	6.098	6.731	0.164	0.165	0.170	−0.804	3.300	yes
2.000	26.000	5.797	5.676	0.173	0.131	0.098	−0.555	-34.000	no
3.000	36.000	3.937	4.060	0.254	0.164	0.172	−0.749	4.100	yes
4.000	23.000	5.618	4.910	0.178	0.175	0.145	−0.846	-20.600	yes
5.000	86.000	4.348	4.920	0.230	0.287	0.308	−0.945	6.800	yes
6.000	26.000	5.618	5.830	0.178	0.219	0.231	−0.915	5.200	yes
7.000	27.000	3.521	4.420	0.284	0.205	0.262	−0.724	21.700	yes
8.000	54.000	2.833	2.760	0.353	0.256	0.290	−0.824	11.700	yes
9.000	32.000	8.953	9.720	0.112	0.156	0.149	−0.919	-4.400	yes
10.000	60.000	2.480	5.330	0.403	0.261	0.282	−0.926	7.400	yes
11.000	39.000	1.790	1.920	0.559	0.292	0.369	−0.844	20.700	yes
12.000	33.000	3.789	2.510	0.264	0.171	0.160	−0.742	-6.300	yes
13.000	50.000	5.879	5.410	0.170	0.139	0.140	−0.730	0.700	yes
14.000	16.000	7.452	8.780	0.134	0.084	0.018	0.127	-370.000	no
15.000	15.000	7.402	6.410	0.135	0.143	0.039	−0.412	-268.000	no
16.000	14.000	8.576	9.330	0.117	0.129	0.052	−0.435	-148.000	no
17.000	15.000	9.257	10.590	0.108	0.077	−0.035	0.348	-119.000	no
18.000	28.000	5.045	4.810	0.198	0.122	0.115	−0.549	-6.500	yes
19.000	15.000	3.928	3.250	0.255	0.106	0.083	−0.667	-120.000	no
20.000	22.000	4.108	3.880	0.243	0.174	0.106	−0.706	-63.500	no
21.000	27.000	5.353	5.090	0.187	0.193	0.225	−0.772	14.400	yes
22.000	31.000	4.713	4.780	0.212	0.100	0.082	−0.270	-22.300	no
23.000	49.000	3.070	3.080	0.326	0.288	0.306	−0.841	5.900	yes
24.000	15.000	9.470	8.440	0.106	0.050	0.193	0.726	73.900	no
25.000	23.000	6.498	6.030	0.154	0.156	0.123	−0.801	20.960	yes
26.000	39.000	5.131	4.580	0.195	0.207	0.206	−0.890	−0.400	yes
27.000	98.000	4.223	4.370	0.237	0.244	0.247	−0.947	1.200	yes
28.000	12.000	6.998	5.620	0.143	0.153	0.030	−0.322	-428.000	no
29.000	16.000	7.559	6.410	0.132	0.207	0.100	−0.856	-107.000	no
30.000	87.000	5.470	5.120	0.183	0.151	0.156	−0.897	3.100	yes
31.000	15.000	2.007	1.990	0.498	0.190	0.118	−0.708	-60.000	no

Fig. A.39 – Results of the fit by least squares to an equation of the form $y = ae^{-bx}$

No. of the traverse	No. of observation	Calculated Chi^2	Critical Chi^2 $_{0.05,\mu}$	Acceptance
1.000	151.000	5.453	19.675	yes
3.000	36.000	3.331	9.488	yes
4.000	23.000	0.430	7.815	yes
5.000	86.000	10.677	15.507	yes
6.000	26.000	0.346	7.815	yes
7.000	27.000	2.885	7.815	yes
8.000	54.000	4.451	11.071	yes
9.000	32.000	5.033	9.488	yes
10.000	60.000	7.560	11.071	yes
11.000	39.000	6.706	9.488	yes
12.000	33.000	7.508	9.488	yes
13.000	50.000	1.830	12.592	yes
21.000	27.000	1.430	9.488	yes
23.000	49.000	6.853	11.071	yes
25.000	23.000	0.836	7.815	yes
26.000	39.000	2.801	12.592	yes
27.000	98.000	3.758	14.067	yes
30.000	87.000	8.700	18.307	yes

Fig. A.40 – Results of the Chi–square test carried out on the traverses that were not rejected

b) Results of the Verification of the Negative Exponential Distribution

Of the 31 traverses described in the table in Figure A.39, 18 were compatible with the negative exponential distribution as a function of the acceptance criterion suggested, that is,

$$\frac{b-a}{a} \leq 0.25$$

but also taking into account a coefficient of correlation r that was sufficiently close to 1.0.

We can see that for 10 of the 18 traverses that were not rejected, the value of parameter λ (estimated by $1/\bar{x}$) is very close to those of a and b, which is consistent with the theoretical distribution law. For the eight others, these values were significantly different, and the Chi-square test was used to better analyse the situation.

All the results of the Chi-square test shown in Figure A.40 would lead to an acceptance of the hypothesis formulating a negative exponential law, which tends to confirm that the scatter observed for these eight traverses was not significant, and that the joint spacings observed in igneous rocks of the Canadian Shield obey the distribution law proposed. Consequently, RQD can be estimated by the Priest and Hudson formula in this latter environment.

We can make a final observation regarding the table in Figure A.39. With the exception of three traverses where we had 26, 26, and 31 spacing values respectively, all the rejected traverses had a number of observations that were too low, from 12 to 16. Thus, it may be that the reason why 11 traverses had to be rejected is that the number of observations were too low. Consequently, we will recommend that the length of the traverses established along the wall should always be enough to allow the observation of at least 30 intersecting fractures.

c) Confidence Interval on the Calculated RQD

According to the central-limit theorem, regardless of the distribution law governing a series of observations, the distribution of the random variable $\bar{x}$ (average value of these observations) may be approximated by a normal law, provided that the size of the sample is large enough ($N \geq 30$), and that we know the mean and variance of the sample. Under these conditions, the confidence interval for ($\bar{x}$) for N spacing observations can be obtained by:

$$\bar{x} \pm \bar{x} \cdot \frac{Z_{\alpha/2}}{\sqrt{N}}$$

where $Z_{\alpha/2}$ is the variable of a normal distribution reduced to a confidence level $1 - \alpha$. For example, for $\alpha = 0.1$ (90% confidence), and 30 observations ($N = 30$), the real value of $\bar{x}$ would be in the interval $\bar{x} \pm 0.3\ \bar{x}$.

This property was used to establish the confidence interval that applies to an RQD calculated by the Priest and Hudson equation shown at the beginning of this appendix, with the addition of the confidence interval on the value of $\bar{x}$ ($=1/\lambda$), for various values of the size of the sample that are higher than 30. These calculations are summarized in the graph shown in Figure A.41.

An example of how this graph is used is given below.

Length of the traverse: 15 m

Number of fractures surveyed: 60

Number of fractures per metre: $\lambda = \dfrac{60}{15} = 4$

$\text{RQD} = 100.\ (0.1\ x\ 4 + 1).e^{-0.1\ x\ 4} = 93.8\%$

Confidence interval for the RQD (Figure A.41): + 4.8%

Thus: $89\% \leq \text{RQD} \leq 98.6\%$

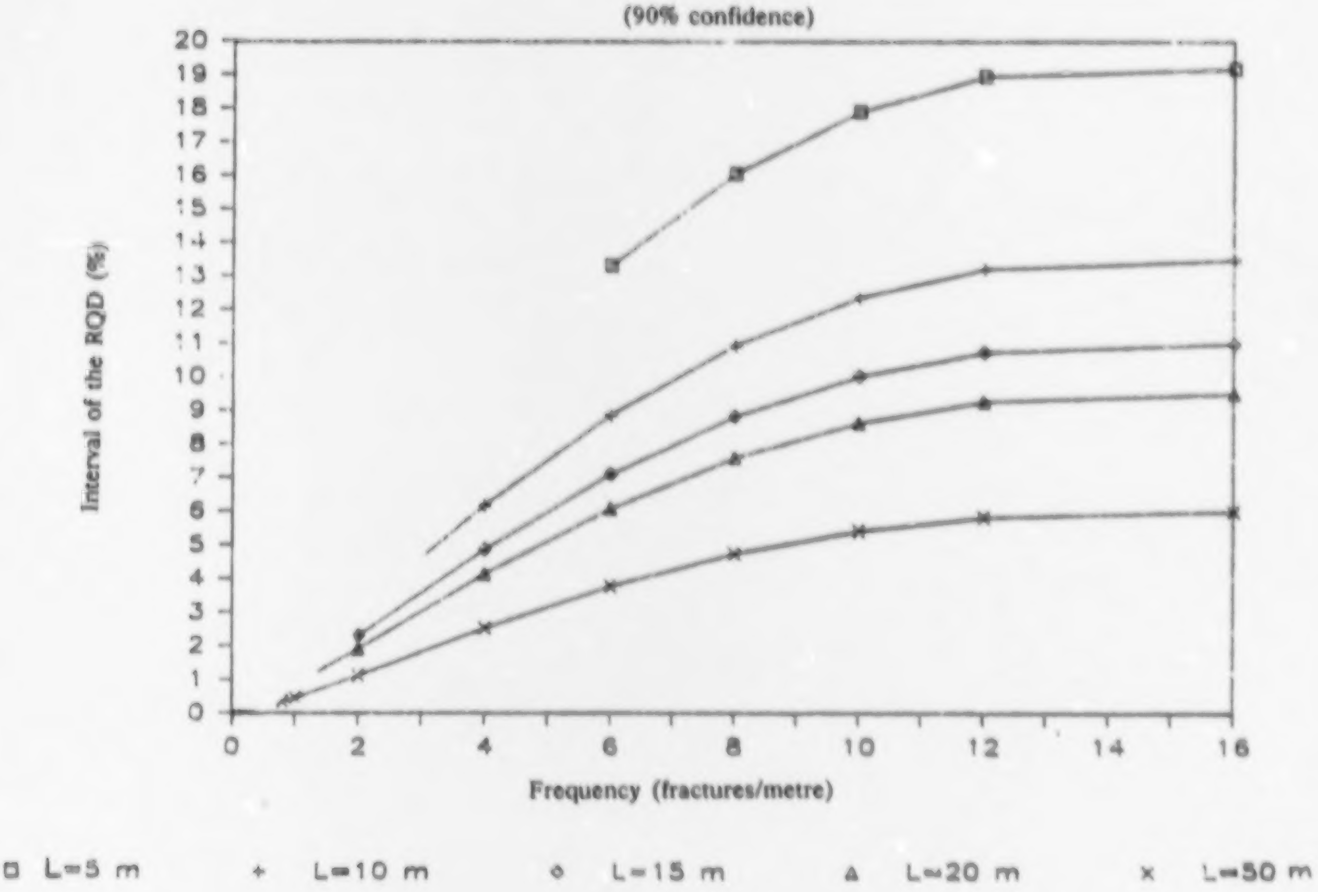


Fig. A.41 – Confidence interval for the RQD as a function of the number of fractures per metre (L: length of the traverse line)

A.21 – ROCK BOLTING MEASURING CAMPAIGN CARRIED OUT IN 10 QUEBEC MINES IN 1985

A rock bolting measuring campaign was carried out in 57 sections of gallery measuring 15 m long in 10 underground mines by Charette and the author (17, 18), to evaluate the quality of the grounds based on the most commonly used geomechanics classifications, and then to compare them with the density of support observed on site.

The following observations were collected for each gallery section:

- dimensions (height and width);
- depth under surface;
- uniaxial compressive strength σ_c : evaluated using a type L Schmidt hammer (Section 4.2.1.2, and Appendix 19);
- RQD: evaluated by counting the number of fractures longer than 30 cm surveyed along a horizontal traverse at the wall (Section 4.2.2.1, and Appendix 20).
- density of bolting: evaluated by counting the number of bolts in the roof and eventually in the wall. The density of bolting was expressed in bolts/m^2, taking into account the surface of the roof and walls where they were found.

The galleries surveyed during the campaign are a representative sample of the conditions encountered in most Quebec mines. Graphs shown in Figure A.42 summarize the measures obtained. Depth of the galleries was from 20 m to 1,000 m; width of the galleries was 2.7 m to 7.3 m, altho[illegible] most were under 5 m. The fracturation densities differed widely, from 0.4 to 5 fractures per metre; the compressive strengths were between 35 and 340 MPa.

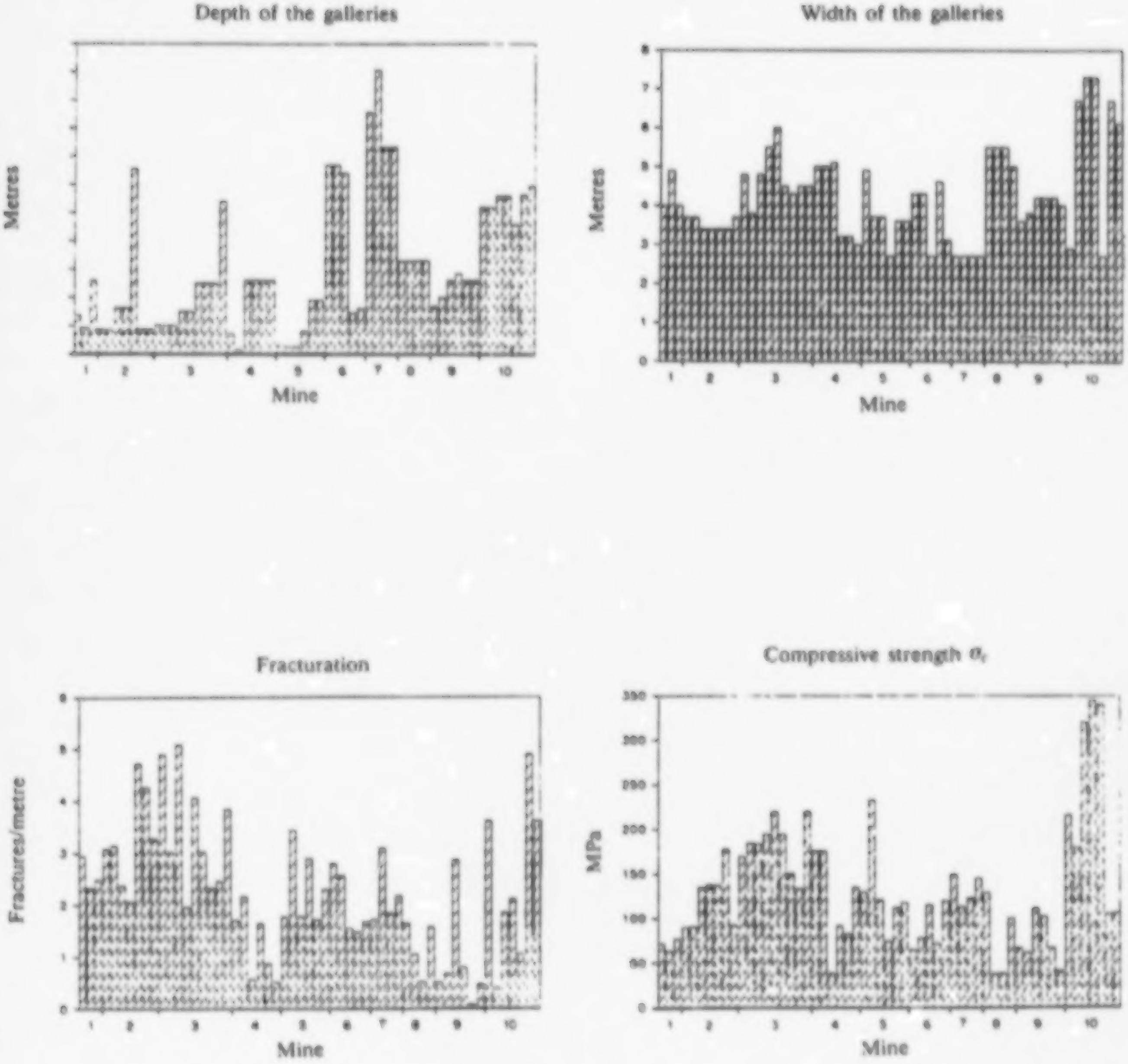


Fig. A.42 – Results of the measures and tests carried out on 57 gallery sections 15 m long (18)

All of these parameters were used to evaluate the most commonly used geomechanics classifications, which are:

- the CSIR classification (69);
- the CSIR classification with the Laubscher and Taylor adjustments (73);
- the Laubscher classification (71) with the Laubscher and Taylor adjustments (73);
- the NGI classification (70);
- the Brook and Dharmaratne classification (A.23), also known as the SRMR (Simplified Rock Mass Rating) classification.

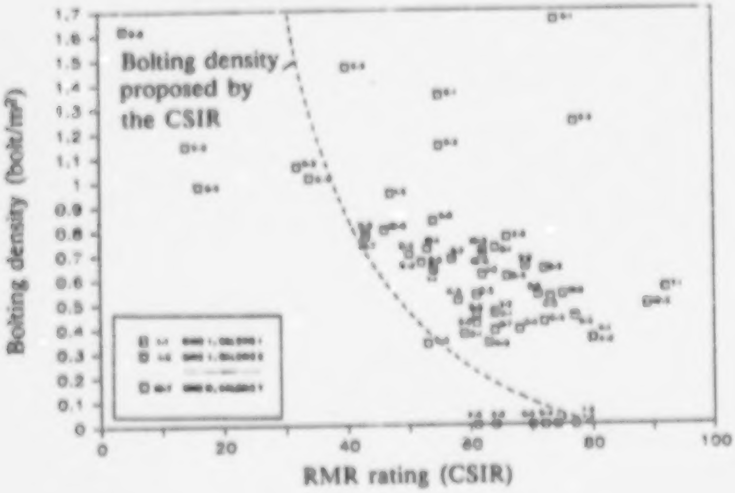
The SRMR is a simplified classification that includes only the sum of the following four parameters: uniaxial compressive strength, joint spacing weighted by the number of joint sets, type of joint, and water in-flows. It seemed useful to test this classification to compare its predictions and performance with those of the other classifications.

The five geomechanics classifications were evaluated separately for the 57 gallery sections. The results of these evaluations are shown in the graph in Figure A.43 as a function of the density of bolting observed in the same location. A curve drawn in broken lines was added to the graph. This corresponds to the required density of bolting suggested by the authors of each of the geomechanics classifications. This density of bolting was obtained from the following references:

- the CSIR classification, (69 or A.23);
- the CSIR classification with the Laubscher and Taylor adjustments (73 or A.1, p. 299);
- the NGI classification (70 or A 1);
- the Laubscher classification with the Laubscher and Taylor adjustments (73 or A.1, p. 290–295);
- the Brook and Dharmaratne classification (73 or A.1, p. 299 or A.23).

The bolting densities of the second, fourth, and fifth classifications were obtained from the same source; that is, the Laubscher and Taylor recommendations (73). The bolting densities of the two other classifications are from a source completely independent of Laubscher or Taylor.

Bolting density vs RMR (CSIR)



Bolting density vs RMR (L & T adjustments)

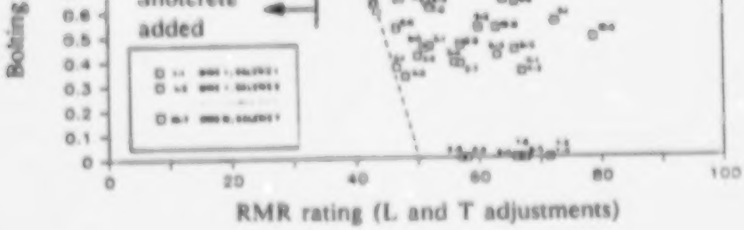
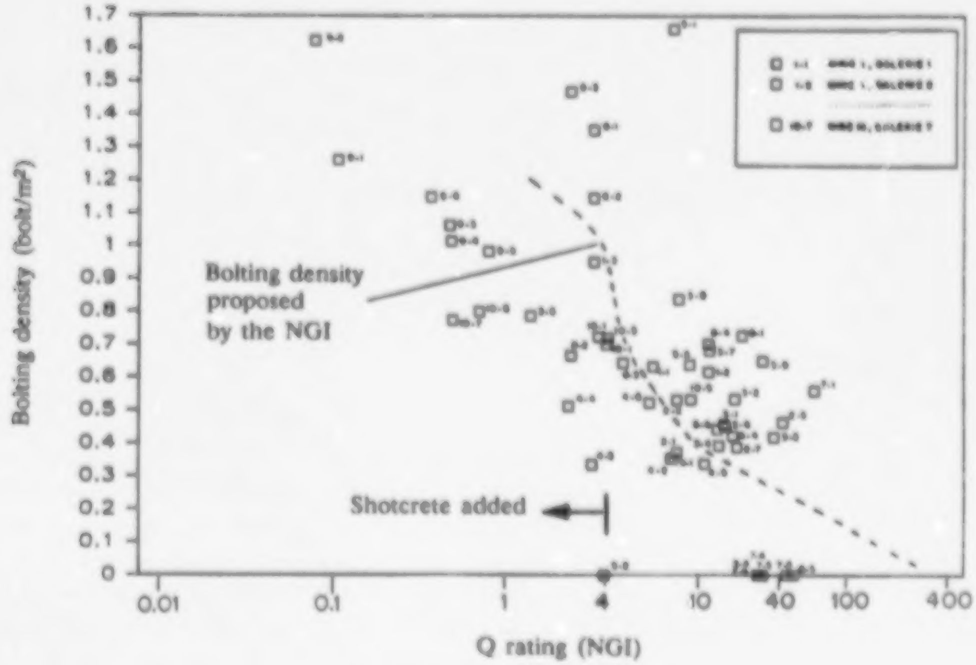


Fig. A.43 - Density of bolting as a function of the five most commonly used geomechanics classifications evaluated in 57 gallery Sections 15 m long (18)

Bolting density vs Q (NGI)



Bolting density vs R Laubscher (L & T adjustments)

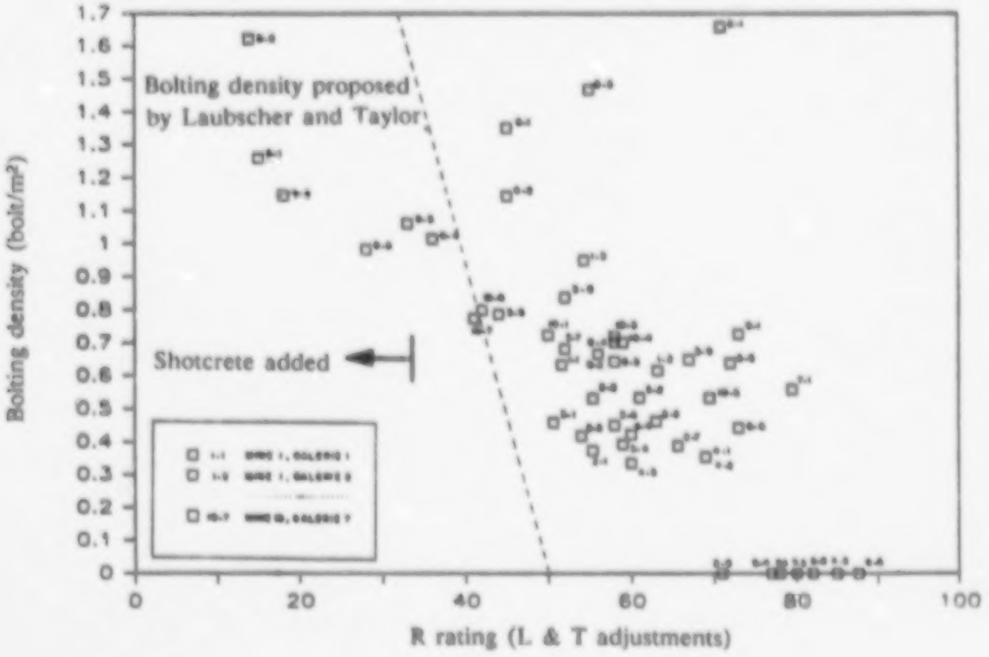


Figure A.43: (cont'd)

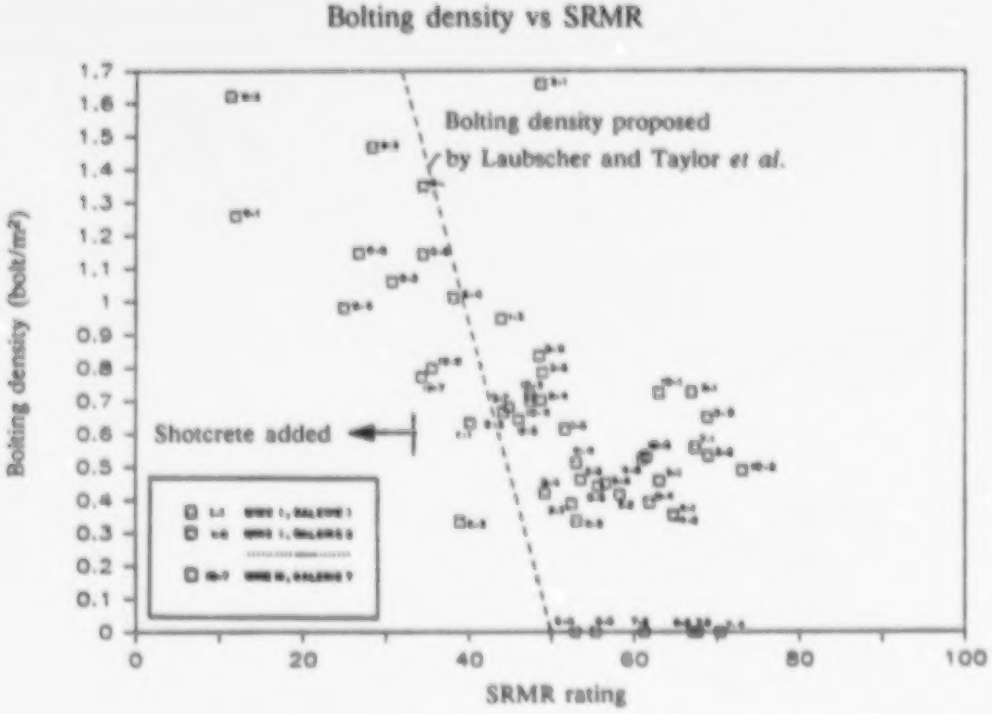


Figure A.43: (cont'd)

The following observations can be derived from an examination of the graph in Figure A.43:

- Except for mine 9, most of the galleries visited seemed to be located in grounds of intermediate quality, since most of the ratings calculated can be grouped around the middle of the scale in the various classifications.

- The point distributions of all the classifications were similar. They were characterized by a concentration in the centre of the graph, which indicates that for almost identical ground qualities, the bolting density used varies by a factor of 1 to 2 or even more, depending on the mine where the observation was made.

- Because the NGI formula is based on multiplication of factors (see Appendix 22), and the other classifications are based on addition of factors, the ratings obtained with the NGI were put on a logarithmic scale, so that the graph could be compared with graphs of the other classifications. Since the NGI Q rating can vary from 0.001 to 1,000, the values of 0.1 to 50 shown on the graph confirm that the galleries visited are primarily located in grounds of intermediate quality.

- It seems that the density of bolting found in mining galleries is almost always higher, sometimes much higher, than the density of support proposed by the authors of the geomechanics classifications. The only exception is for the NGI classification, where the number of points on each side of the recommended bolting density are almost equal.

- The broken line representing the Laubscher and Taylor bolting recommendation, and to a lesser extent the CSIR line, have a very pronounced slope, so that the density of support may be doubled for a very slight variation of only a few points in the geomechanics classification. This heavily slanted line is not compatible with the trend of the points on the various graphs where support density increases more slowly, as a function of the value of the geomechanics classification. For this reason, the slopes of the lines representing the support densities recommended further on in this appendix will be less pronounced.

- The types of bolts found in the galleries are shown in Figure A.44. Most of the bolts were mechanically anchored; nevertheless, 11 galleries had resin-grouted bolts, and 6 galleries (in the same mine) had Swellex friction bolts. If we represent the points corresponding to the latter two types of bolts on the graphs in Figure A.43, we can find no trend regarding the type of ground where they are used. They are found in grounds of poor quality (low geomechanics classification rating) and also in grounds with much higher ratings.

Seven gallery sections visited had no support. This observation is useful when we compare the various geomechanics classifications to determine which of them makes it possible to arrive at the best predictions of no support needed. An examination of the graphs in Figure A.43 shows an anomaly common to the CSIR, the CSIR with the Laubscher and Taylor adjustments, and the SRMR classifications: several points corresponding to supported galleries are above the seven points representing unsupported galleries. This observation means that the three geomechanics classifications mentioned do not make it possible to discriminate between galleries that need support and those where support is not necessary, even though this is one of the main reasons to use geomechanics classifications. The Laubscher and NGI classifications are much more successful in reaching this objective. If we examine the corresponding graphs, the boundary between supported and unsupported galleries could be set at 75 for the Laubscher classification, and at 30 for NGI. However, we should point out that the unsupported galleries visited were quite narrow in width (between 2.7 and 3.5 m), except for one gallery that was 5 m wide.

Mine-gallery	Type of bolt
1-1	mechanically anchored
1-2	mechanically anchored
1-3	mechanically anchored
2-1	mechanically anchored
2-2	mechanically anchored
2-3	mechanically anchored
2-4	mechanically anchored
2-5	mechanically anchored
2-6	no bolting
2-7	mechanically anchored
3-1	mechanically anchored
3-2	mechanically anchored
3-3	mechanically anchored
3-4	mechanically anchored
3-5	mechanically anchored
3-6	mechanically anchored
3-7	mechanically anchored
3-8	mechanically anchored
3-9	mechanically anchored
4-1	mechanically anchored
4-2	mechanically anchored
4-3	mechanically anchored
4-4	mechanically anchored
4-5	mechanically anchored
4-6	mechanically anchored
5-1	resin-grouted
5-2	no bolting
5-3	resin-grouted
5-4	no bolting
5-5	resin-grouted
5-6	resin-grouted
6-1	mechanically anchored and grouted
6-2	mechanically anchored and grouted
6-3	mechanically anchored and grouted
6-4	mechanically anchored
6-5	no bolting
7-1	mechanically anchored
7-2	no bolting
7-3	no bolting
7-4	no bolting
8-1	resin-grouted
8-2	resin-grouted
8-3	resin-grouted
8-4	resin-grouted
9-1	friction (swellex)
9-2	friction (swellex)
9-3	friction (swellex)
9-4	friction (swellex)
9-5	friction (swellex)
9-6	friction (swellex)
10-1	mechanically anchored
10-2	mechanically anchored
10-3	mechanically anchored
10-4	mechanically anchored
10-5	mechanically anchored
10-6	mechanically anchored
10-7	mechanically anchored

Fig. A.44 – Types of bolts found in 57 gallery sections 15 m long (18)

- If we take into account the observations made in the previous conclusion, the two classifications that could be recommended for an evaluation of ground conditions in a gallery are the Laubscher and NGI classifications. Moreover, an initial evaluation of the bolting density required could be obtained using one of the following two formulas:

$$\left| \begin{array}{l} D = -0.0214\ R + 1.68 \\ D = -0.227 \ln Q + 0.839 \end{array} \right.$$

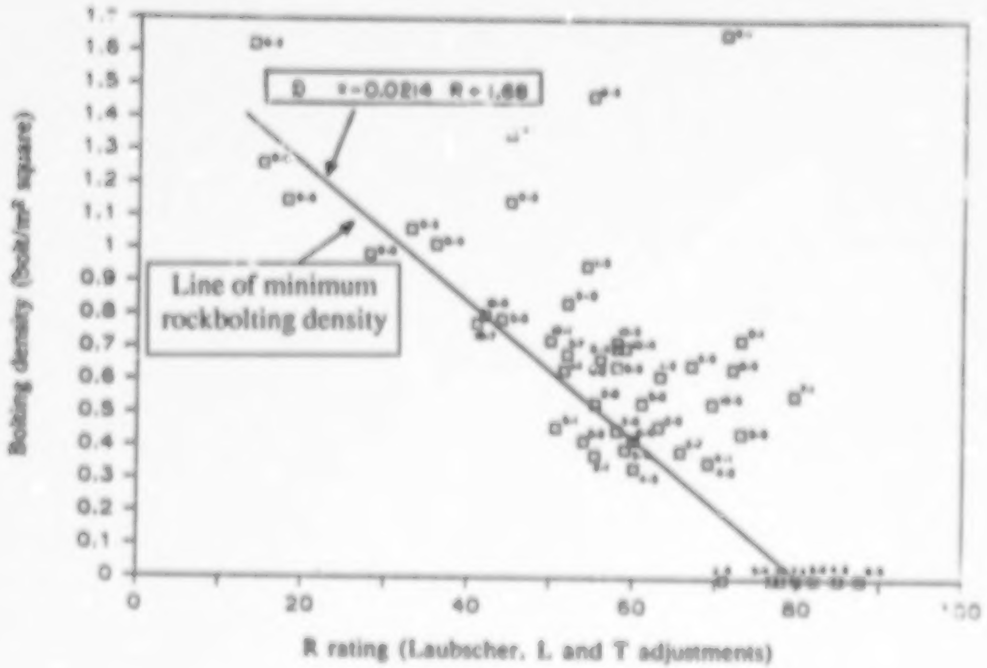
where D: density of bolting (bolts/m^2 of roof, and wall if necessary)

R: rating obtained with the Laubscher geomechanics classification with the Laubscher and Taylor adjustments

Q: rating obtained with the NGI geomechanics classification

These two formulas were obtained by superimposing on the graphs of the two classifications shown in Figure A.45 a line representing the support observed in the galleries. The two lines were deliberately placed at the base rather than in the middle of the point scatter. Their function is to take into account the bolting densities effectively found in the galleries, since these would correspond to a minimum degree of satisfaction on the part of the miners and supervisors involved in their construction.

Bolting density vs R Laubscher (L & T adjustments)



Bolting density vs Q(NGI)

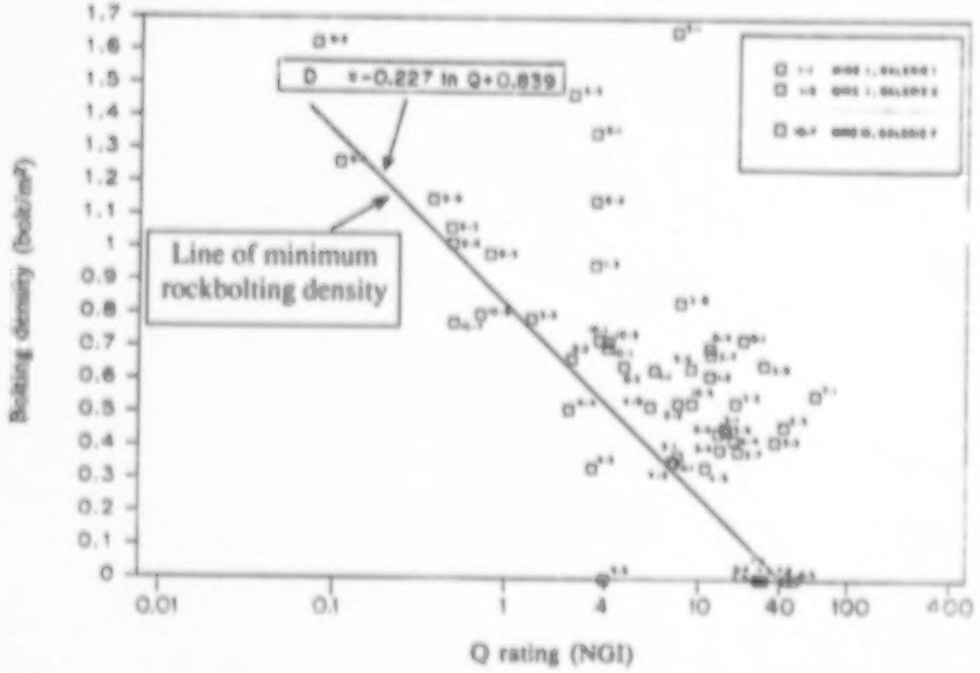


Fig. A.45 - Superimposition of lines representing the minimum bolting density in the galleries as a function of the Laubscher and NGI geomechanics classifications

A.22 – TABLES USED FOR THE DETERMINATION OF THE GEOMECHANICS CLASSIFICATION RATINGS OF ROCK MASSES

a) CSIR Classification (A.1, p. 26–27)

This rating is obtained by the following formula:

RMR = Parameter 1 + Parameter 2 + Parameter 3 + Parameter 4 + Parameter 5

– adjustment for the joint orientation (Table B)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	> 8 MPa	4 - 8 MPa	2 - 4 MPa	1 - 2 MPa	For this low range "uniaxial compres." sive test is preferred		
		Uniaxial compressive strength	> 200 MPa	100 - 200 MPa	50 - 100 MPa	25 - 50 MPa	10-25 MPa	3-10 MPa	1-3 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		> 3 m	1 - 3 m	0.3 - 1 m	50 - 300 mm	< 50 mm		
	Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Soft joint wall rock	Slickensided surfaces or Gouge < 5 mm thick or Joints open 1 - 5 mm Continuous joints	Soft gouge > 5 mm thick or Joints open > 5 mm Continuous joints		
	Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		< 25 litres/min	25 - 125 litres/min	> 125 litres/min		
		OR Ratio joint water pressure / major principal stress	0		0.0 - 0.2	0.2 - 0.5	> 0.5		
		OR General conditions	Completely dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Rating		10		7	4	0		

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100—81	80—61	60—41	40—21	< 20
Class No	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 5 m span	6 months for 4 m span	1 week for 3 m span	5 hours for 1.5 m span	10 min for 0.5 m span
Cohesion of the rock mass	> 300 kPa	200-300 kPa	150-200 kPa	100-150 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	40°-45°	35°-40°	30°-35°	< 30°

TABLE 6 - THE EFFECT OF JOINT STRIKE AND DIP ORIENTATIONS IN TUNNELLING

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0°-20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

Fig. A.46 – CSIR Classification. From Bieniawski (69)

The correction for joint orientation is easy to determine when a joint set is predominant in relation to the others. When there are several joint sets, however, it is preferable not to use this correction and to use instead the Laubscher and Taylor adjustments as shown below.

b) Laubscher and Taylor Adjustments to the CSIR Classification (A.1, pp. 297 – 298)

These adjustments apply to parameters 1, 2, 3, and 4 of the CSIR classification and are concerned with the following effects: weathering, field and induced stresses, changes in stress, influence of strike and dip orientations, and blasting effects.

The adjustments are obtained on the basis of the guidlines presented in Figure A.47.

The adjustments are made by modifying the original value of a particular parameter by a percentage determined by the influence of the various effects on that parameter. For example, if an RQD value of 75 is established from an examination of diamond drill core and it is known that the blasting techniques to be used for mining in this rock are poor, the RQD is reduced by 20% to give an adjusted value of 60.

Also, some parameters may be affected by several corrections. For example, if parameter 4 of the CSIR classification has an initial value of 12, and all corrections apply, the final value becomes:

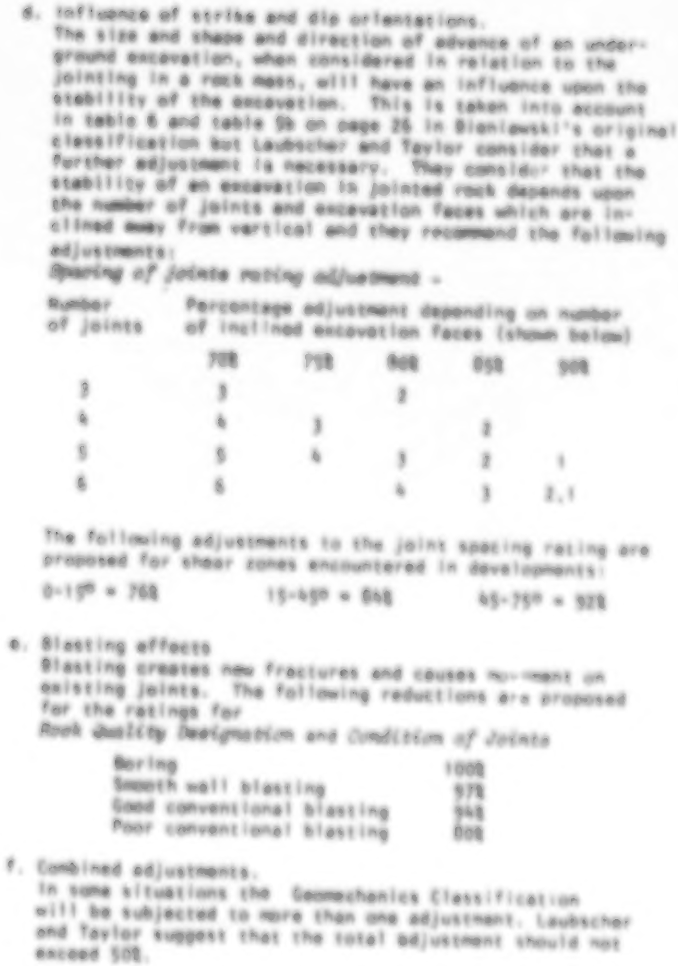
$$12 \times 82\% \times 76\% \times 60\% \times 80\% = 3.58$$

We should emphasize that paragraph f (Combined adjustments) in Figure A.47 states that the total correction to the RMR rating should not reduce its initial value by more than 50%.

a. Weathering.
Certain rock types weather rapidly on exposure and this aspect must be taken into account in deciding upon permanent support measures. Weathering affects three of the parameters listed in table 5 on page 26:
Intact rock strength - decrease by up to 96% if weathering takes place along micro-structures in the rock.
Rock Quality Designation - decrease by up to 95% as the rock weathers resulting in an increase in fractures.
Condition of joints - Reduce the rating for the condition of joints by up to 82% if weathering is considered to cause deterioration of the joint wall rock or the joint filling.

b. Field and induced stresses.
Field and induced stresses can influence the condition of joints by keeping the joint surfaces in compression or by allowing joints to loosen and hence increase the possibility of shear movement.
Condition of joints - when the stress conditions are such that joints will be kept in compression, increase the rating by up to 120%. If the possibility of shear movement is increased, decrease the rating by up to 90%. If the joints are open and can be equated to joints with thin gouge filling, decrease the rating by up to 76%.

c. Changes in stress.
When large stress changes are induced by mining operations, for example during the extraction of crown pillars or the over-mining of draw-points, the condition of the joints will be changed in the same way as in b. above.
Condition of joints - when stress changes are such that joints will always be in compression, increase rating by up to 120%. When stress changes are likely to cause serious shear movement or joint opening, decrease rating by up to 60%.



d. Influence of strike and dip orientations.
The size and shape and direction of advance of an underground excavation, when considered in relation to the jointing in a rock mass, will have an influence upon the stability of the excavation. This is taken into account in table 6 and table 5b on page 26 in Bieniawski's original classification but Laubscher and Taylor consider that a further adjustment is necessary. They consider that the stability of an excavation in jointed rock depends upon the number of joints and excavation faces which are inclined away from vertical and they recommend the following adjustments:

Spacing of joints rating adjustment -

Number of joints	Percentage adjustment depending on number of inclined excavation faces (shown below)				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6		4	3	2,1

The following adjustments to the joint spacing rating are proposed for shear zones encountered in developments:

0-15° = 76%　　15-45° = 84%　　45-75° = 92%

e. Blasting effects
Blasting creates new fractures and causes movement on existing joints. The following reductions are proposed for the ratings for
Rock Quality Designation and *Condition of Joints*

Boring	100%
Smooth wall blasting	97%
Good conventional blasting	94%
Poor conventional blasting	80%

f. Combined adjustments.
In some situations the Geomechanics Classification will be subjected to more than one adjustment. Laubscher and Taylor suggest that the total adjustment should not exceed 50%.

Fig. A.47 – Laubscher and Taylor adjustments to the CSIR Classification. From Laubscher and Taylor (73)

c) NGI Classification (A.1, p. 31–33)

This rating is obtained from the following formula:

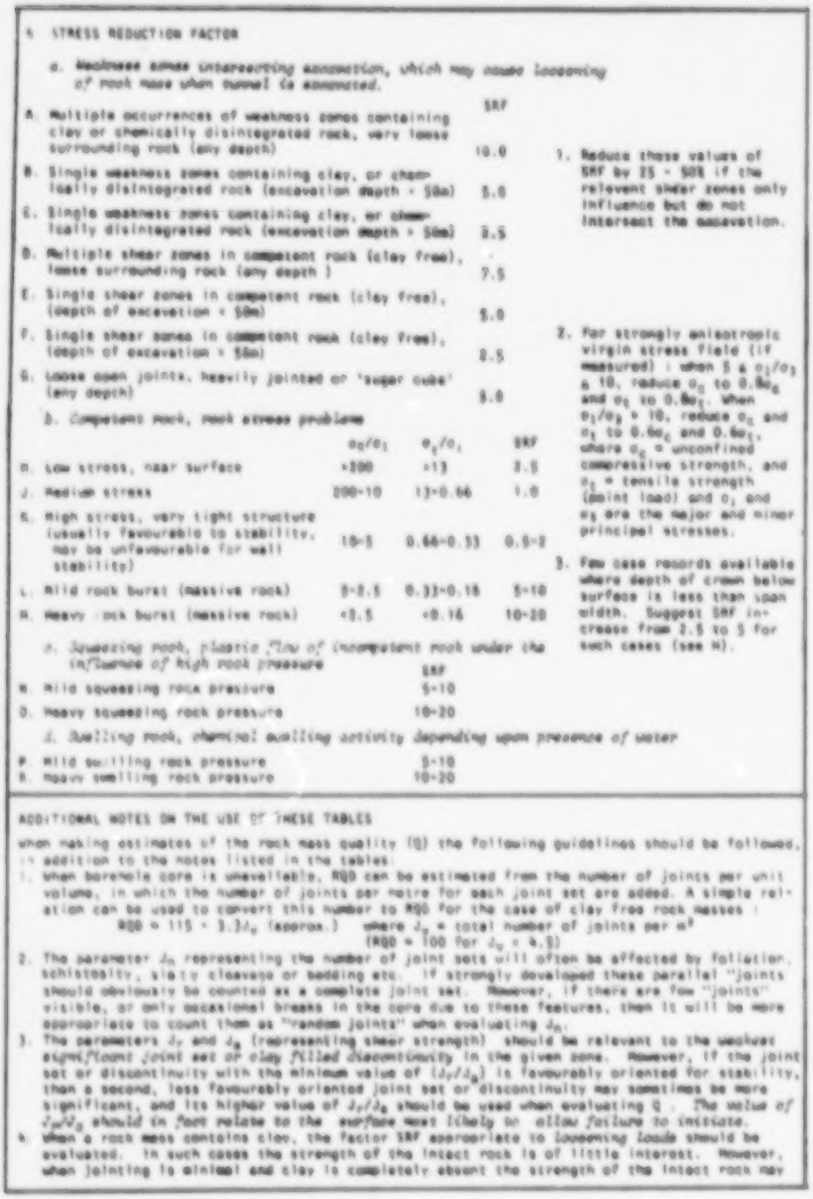
$$Q = \frac{RQD}{Jn} \cdot \frac{Jr}{Ja} \cdot \frac{Jw}{SRF}$$

The various parameters can be evaluated with the help of the following table:

Description	Value	Notes
1. ROCK QUALITY DESIGNATION	RQD	
A. Very poor	0 - 25	1. Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate.
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use $(3.0 \times J_n)$
G. Three joint sets plus random	12	2. For portals use $(2.0 \times J_n)$
H. Four or more joint sets, random, heavily jointed 'sugar cube', etc	15	
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact and		
b. Rock wall contact before 10 cms shear.		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensided, undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength.
G. Slickensided, planar	0.5	
c. No rock wall contact when sheared.		
H. Zone containing clay minerals thick enough to prevent rock wall contact.	1.0	
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact.	1.0	
4. JOINT ALTERATION NUMBER	J_a	ϕ_r (approx.)
a. Rock wall contact.		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	—

	J_a	ϕ_r (approx.)	
B. Unaltered joint walls, surface staining only	1.0	(25° - 35°)	
C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	(25° - 30°)	1. Values of ϕ_r, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20° - 25°)	
E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2mm or less in thickness)	4.0	(8° - 16°)	
b. Rock wall contact before 10 cms shear.			
F. Sandy particles, clay-free disintegrated rock etc	4.0	(25° - 30°)	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick)	6.0	(16° - 24°)	
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick)	8.0	(12° - 16°)	
J. Swelling clay fillings, i.e. montmorillonite (continuous, < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water	8.0 - 12.0	(6° - 12°)	
c. No rock wall contact when sheared.			
K. Zones or bands of disintegrated	6.0		
L. or crushed rock and clay (see	8.0		
M. G,H and J for clay conditions)	8.0 - 12.0	(6° - 24°)	
N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening)	5.0		
O. Thick, continuous zones or			
P. bands of clay (see G, H and	10.0 - 13.0	(6° - 24°)	
R. J for clay conditions)	13.0 - 20.0		

5. JOINT WATER REDUCTION FACTOR	J_w	approx. water pressure (kgf/cm²)	
A. Dry excavations or minor inflow, i.e. < 5 litre/min. locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
D. Large inflow or high pressure , considerable outwash of fillings	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure continuing without decay	0.1 - 0.05	> 10	



6. STRESS REDUCTION FACTOR

a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.

	SRF
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10.0
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50m)	5.0
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50m)	2.5
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5
E. Single shear zones in competent rock (clay free), (depth of excavation < 50m)	5.0
F. Single shear zones in competent rock (clay free), (depth of excavation > 50m)	2.5
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)	5.0

b. Competent rock, rock stress problems

	σ_c/σ_1	σ_t/σ_1	SRF
H. Low stress, near surface	>200	>13	2.5
J. Medium stress	200-10	13-0.66	1.0
K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-0.33	0.5-2
L. Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20

c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure

	SRF
N. Mild squeezing rock pressure	5-10
O. Heavy squeezing rock pressure	10-20

d. Swelling rock, chemical swelling activity depending upon presence of water

	SRF
P. Mild swelling rock pressure	5-10
R. Heavy swelling rock pressure	10-20

1. Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence but do not intersect the excavation.
2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses :

   $RQD = 115 - 3.3J_v$ (approx.) where J_v = total number of joints per m³

   ($RQD = 100$ for $J_v < 4.5$)
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest *significant joint set or clay filled discontinuity* in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q. *The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.*
4. When a rock mass contains clay, the factor SRF appropriate to *loosening loads* should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may

Fig. A.48 – NGI Geomechanics classification. From Barton, Lien and Lunde (70)

The SRF (Stress Reduction Factor) parameter makes it possible to correct the rating in a way that is similar to the correction made to the RMR rating with the Laubscher and Taylor adjustments. This parameter makes it possible to take into account the local effects of relaxation and block loosening caused by the shear zones of weak areas, or to take into account the value of natural stresses acting deeply or near the surface, or the effect of the swelling capacity of the rock.

The authors of the classification also emphasize that persistent schistosity should be considered as a joint set and counted as such (see note 2 in Fig. A.48).

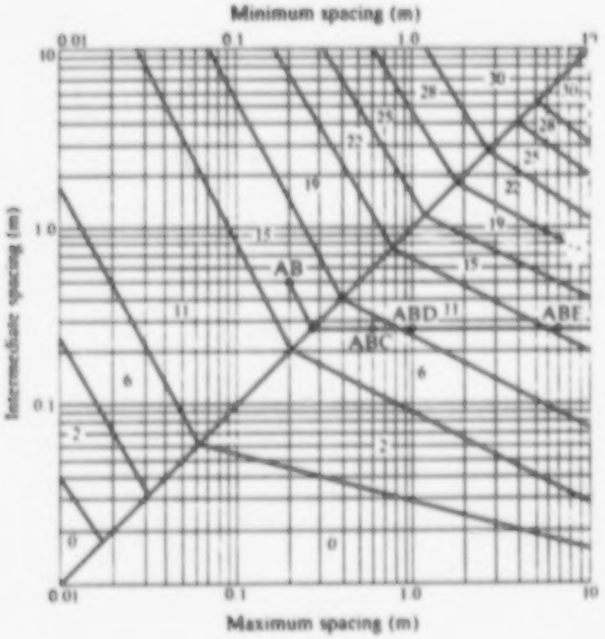
d) Laubscher Classification (A.24, p. 81–84)

This rating can be obtained from the following formula:

$$R = \text{Parameter } 1 + \text{Parameter } 2 + \text{Parameter } 3 + \text{Parameter } 4 + \text{Parameter } 5$$

These parameters can be evaluated from the table below.

	class	1		2		3		4		5		
	rating	100–81		80–61		60–41		40–21		20–0		
	description	very good		good		fair		poor		very poor		
	subclasses	*A*	*B*	*A*	*B*	*A*	*B*	*A*	*B*	*A*	*B*	
	item											
1	*RQD*, %	100–91	90–76	75–66	65–56	55–46	45–36	35–26	25–16	15–6	5–0	
	rating	20	18	15	13	11	9	7	5	3	0	
2	*IRS*, MPa	141–136	135–126	125–111	110–96	95–81	80–66	65–51	50–36	35–21	20–6	5–0
	rating	10	9	8	7	6	5	4	3	2	1	0
3	joint spacing					refer to Figure 3.31						
	rating	30 …………										………… 0
4	condition of joint	45° …………				static angle of friction						………… 5°
						refer to Table 3.9						
	rating	30 …………										………… 0
		inflow per 10 m length			0		$25\ \ell\,\mathrm{min}^{-1}$		$25–125\ \ell\,\mathrm{min}^{-1}$		$125\ \ell\,\mathrm{min}^{-1}$	
		or										
5	groundwater	$\frac{\text{joint water pressure}}{\text{major principal stress}}$			0		0.0–0.2		0.2–0.5		0.5	
		or										
	description	completely dry			completely dry		moist only		moderate pressure		severe problems	
	rating	10			10		7		4		0	



Joint spacing ratings for multi-joint systems. In the example, spacings of sets A, B, C, D and E are 0.2, 0.5, 0.6, 1.0 and 7.0 m, respectively; the combined ratings for AB, ABC, ABD and ABE are 15, 6, 11 and 15, respectively

Parameter	Description	Percentage adjustment
joint expression (large scale)	wavy uni-directional	90–99
	curved	80–89
	straight	70–79
joint expression (small scale)	striated	85–99
	smooth	60–84
	polished	50–59
alteration zone	softer than wall rock	70–99
joint filling	coarse hard-sheared	90–99
	fine hard-sheared	80–89
	coarse soft-sheared	70–79
	fine soft-sheared	50–69
	gouge thickness < irregularities	35–49
	gouge thickness > irregularities	12–23
	flowing material > irregularities	0–11

Assessment of joint condition – adjustments as combined percentages of total possible rating of 30

Figure A.49 – Laubscher classification (71)

In reality, the Laubscher classification is a modification of the CSIR classification. It includes the same five parameters, but there are differences in their evaluation. The first improvement affects joint spacing, which is evaluated more rationally with the Laubscher classification, because the authors suggest that the joint sets should be differentiated first and then allocated a value that takes into account the spacing of each set.

When evaluating the latter parameter, the author recommends taking into account joint sets only when their size is higher than the diameter of the gallery or more than 3 m. Joint sets smaller than 3 m could be counted if their orientation is such that they systematically cut across blocks of rock by intersecting with other joint sets. We should emphasize that smaller joints are nevertheless counted, because they play a role in reducing the value of the first parameter, that is, the RQD.

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